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PREDICTION OF END OF CONSTRUCTION UNDRAINED DEFORMATIONS OF ATC--ETC(U)
JUN 72 R FOOTT, C C LADD   DACW39-71-C-0022

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1. INTRODUCTION

The performance of the Test Sections was not entirely sat-
isfactory from a practical viewpoint. Despite very large berms,

CONTRACT REPORT S-72-3

PREDICTION OF END OF CONSTRUCTION UNDRAINED DEFORMATIONS OF ATCHAFALAYA LEVEE FOUNDATION CLAYS

AD-A040181

by

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under

Contract No. DACW39-71-C-0022 New

with

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CORPS OF ENGINEERS
Vicksburg, Mississippi

for

Department of the Army
New Orleans District, Corps of Engineers
New Orleans, Louisiana

Geotechnical Division
Department of Civil Engineering
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ABSTRACT

Levee construction in south-central Louisiana has proven to be very time consuming and expensive because of excessive movements within the soft, highly plastic, thick deposits of the foundation soils. Field measurements from three well instrumented test sections constructed along the East Atchafalaya Basin Protection Levee (EABPL) have led to the belief that undrained shear strains and long term lateral creep deformations are major factors contributing to the very large settlements of the levees.

This report presents the results of finite element predictions of the end of construction lateral deformations on the floodwayside of the EABPL Test Sections II and III. The M.I.T. computer program used for the analyses, which realistically modeled the actual sequence of construction, requires input of the following soil parameters for the foundation soils:

(1) The initial vertical and horizontal stresses

(2) The undrained shear strength as a function of the orientation of the major principal stress at failure

(3) A bilinear stress-strain relationship, and particularly the undrained shear modulus prior to yielding.

The first step in selection of the soil parameters was a thorough investigation of the stress history of the foundation soils, and especially the in situ maximum past pressure. This study included a detailed consideration of the geology of the area, values of maximum past pressure determined from oedometer tests, the results of field vane tests, and variations in water content and plasticity with depth. A principal conclusion from this work, which proved to be very significant, was the discovery of a highly precompressed zone below El. -45ft.

The stress-strain-undrained strength properties of the foundation clay were investigated via a program of consolidated-undrained triaxial, plane strain, and direct-simple shear tests performed on samples K_o consolidated beyond the in situ stresses. The resulting data were normalized and plotted versus overconsolidation ratio and/or shear stress level. The normalized soil parameters, combined with the stress history developed for the foundation clay and a knowledge of zones of highly stressed soil, were then used to select the soil properties required for the analyses. A significant finding was the fact that the Atchafalaya clays have an unusually low shear modulus at stress levels approaching failure.

Only one set of predictions were made for each of the test sections. The results, presented in Chapter 8, showed good to excellent agreement between the predicted pattern of lateral deformations and the measured field performance. Most significantly, the computer analyses predicted that the largest shear strains would occur in the zone between about El. -45 and -20 ft. It was concluded that the computer model and soil parameters that were developed represent a major advance in understanding the fundamental nature of the long term lateral deformations that have proven so detrimental to the performance of the levees.

FOREWORD

The work described in this report was performed under Contract No. DACW39-71-C-0022 entitled "Research Study on Strength-Deformation Properties of Soft Foundation Clays" between the U.S. Army Engineer Waterways Experiment Station (WES) and the Massachusetts Institute of Technology. The research was sponsored by the U.S. Army Engineer District, New Orleans (NOD). The contract was monitored at WES by Mr. Joseph R. Compton, Chief, Embankment and Foundation Branch.

The original eleven month contract, initiated in September 1970, called for an experimental investigation of the "normalized" soil engineering properties of two levee foundation clays, with particular emphasis on evaluation of undrained strength anisotropy, sample disturbance, and values of K_o. The scope of the work was expanded during the next year to include finite element predictions of the end of construction deformations at the floodwayside of the EABPL Test Sections II and III. This report fulfills that contractual obligation. A second report presents the results of the experimental program conducted on the EABPL and St. Charles Parish Lakefront Levee foundation clays.

The selection of soil parameters, performance of the finite element analyses, comparisions with field data, and preparation of this report was done by Mr. Roger Foott, M.I.T. Fellowship student and doctoral candidate, under the supervision of Dr. Charles C. Ladd, Professor of Civil Engineering. Mr. Charles E. Williams, NSF Traineeship graduate student, also helped with preparation of the report. Dr. John T. Christian, Associate Professor of Civil Engineering, and Mr. Richard M. Simon, NSF Fellowship student, developed the basic computer program FEECON used for the analyses and assisted Mr. Foott in modifying the program to handle yielded zones that later became unyielded.

Finally, the assistance of WES and NOD in supplying copies of their laboratory data and in furnishing prompt and complete information concerning construction details is gratefully acknowledged.

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1. INTRODUCTION

Levee construction in the southern portion of the Atchafalaya Basin Floodway in south-central Louisiana has been in progress for 30 to 35 years (Kaufman and Weaver, 1967)*. Construction has proceeded in stages so that the thick (on the order of 100 ft) deposits of soft alluvial and deltaic highly plastic foundation clays can consolidate and strengthen in order to support subsequent lifts of fill. This procedure has been very time consuming and expensive. Moreover, settlements in excess of 10 to 15 ft have made it impossible to attain the desired levee heights (20 to 25 ft above original grade) along large portions of the floodway. Large lateral movements, thought to be caused by undrained shear deformations and creep of the soft foundation clays, have been a major contributor to the excessive vertical settlements of the levee crown.

The field performance of the levees is being studied in detail via three extensively instrumented Test Sections constructed in 1964-1965 along the East Atchafalaya Basin Protection Levee (EABPL), as described by Kaufman and Weaver (1967) and USCE (1968). Test Section I, with a four ft crown and a height of six ft, had a design factor of safety of 1.1 based on a $\phi = 0$** analysis using UU triaxial compression test data. Test Sections II and III, with a wide crown, a height of about ten ft, and very wide stabilizing berms, had nominal factors of safety of 1.1 and 1.3 based on similar total stress stability analyses. The heights were measured above the crown of the existing levee, which at that time was about 17 feet above original ground.

* The list of references is presented in Appendix B

** The notation is presented in Appendix A

The performance of the Test Sections was not entirely satisfactory from a practical viewpoint. Despite very large berms, both Test Sections II and III suffered extensive lateral deformations with a resulting drop in the crest elevation, such that for practical purposes the embankments could be considered to have failed. These lateral shear deformations, which continued long after the end of construction, were particularly noticeable between El. -20 and -40 ft (ground surface El. = 3 $\pm$ 1 ft MSL). These continued lateral deformations are probably caused by creep, which is the subject of separate investigations. However, they are probably strongly influenced by the end of construction patterns of deformation and areas of high stress level. Test Section I was designed as a possible method of raising the levee crest quickly to an interim height. This section suffered a stability failure.

This report presents the results of an investigation of the end-of-construction deformation behavior of Test Sections II and III, culminating in the development of finite element representations of these Test Sections. The study used many of the normalized soil parameter concepts developed by Ladd (1971) and involves the following topics:

(1) Measured performance of Test Sections II and III
(2) Geology of the Test Section area
(3) Stress history of the Test Section foundation soils
(4) Evaluation of soil properties and selection of soil parameters
(5) Ammendment of an existing finite element computer program (FEECON) to allow representation of the embankment construction sequence
(6) Development of the finite element grid and appropriate input of soil parameters
(7) Computer runs for both Sections
(8) Evaluation of the results
(9) Recommendations for further study.

These topics are considered in detail in the following sections.

2. MEASURED PERFORMANCE OF TEST SECTIONS II AND III

The performance of the embankments is described by Kaufman and Weaver (1967), from which all figures in this section have been taken.

The Test Sections were constructed over the existing levees as shown in Fig. 2-1. They were in excess of 500 ft wide, the very extensive flat berms being required for stability. The construction sequence is shown in Figs. 2-1, 2-2, and 2-4.

The Test Sections were very thoroughly instrumented. Typical movements and pore pressures measured at both Sections during and after construction are shown in Figs. 2-2 through 2-5. The most significant features of the performance are:

(1) Very large settlements at the centerline, which occurred during and after placement of the embankment crest material

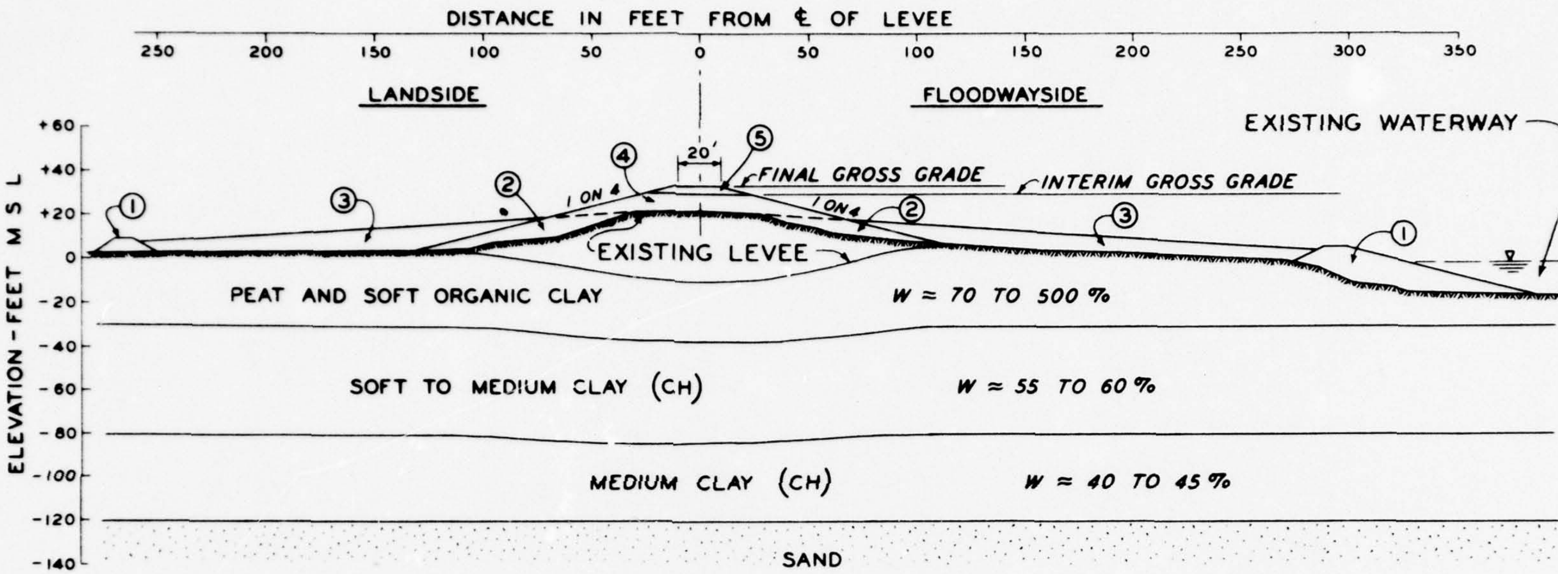
(2) Relatively little decrease in pore pressures during the year after construction

(3) Large lateral deformations, particularly to the floodwayside of the centerline, with continuing large creep deformations occurring after the end of construction

(4) Most of the centerline settlement occurred above El. -60 Most of the lateral deformations also occurred above this elevation. Further, the zone of soil between El. -20 and -40 generally experienced the largest lateral shear deformations, both during and after construction, particularly to the floodwayside.

The above comments apply to both Sections II and III, although the deformations were generally larger for Section II which had smaller berms and a lower factor of safety. A likely interpretation of this behavior is that lateral deformations away from the centerline caused large settlements of the crest such that the design was not entirely satisfactory. Certainly, the relatively small decrease in foundation pore pressures suggests that consolidation had not caused all of the observed

settlements. Thus it seems that the large lateral deformations are the most important characteristic of this performance. Indeed the highly stressed zone between El. -20 and -40, in which much of the creep deformation occurred, is probably the single feature which has most influenced the embankment behavior.



LEGEND

① CAST FILL DIKE

② & ④ SEMI-COMPACTED HAULED FILL

③ HYDRAULIC FILL BERM

⑤ HAULED FILL PLACED IN THIN LIFTS

NOTE: *ENCIRCLED NUMBERS REPRESENT SEQUENCE OF CONSTRUCTION*

FIG. 2-1 PLAN FOR RAISING LEVEE GRADE AND TYPICAL FOUNDATION CONDITIONS

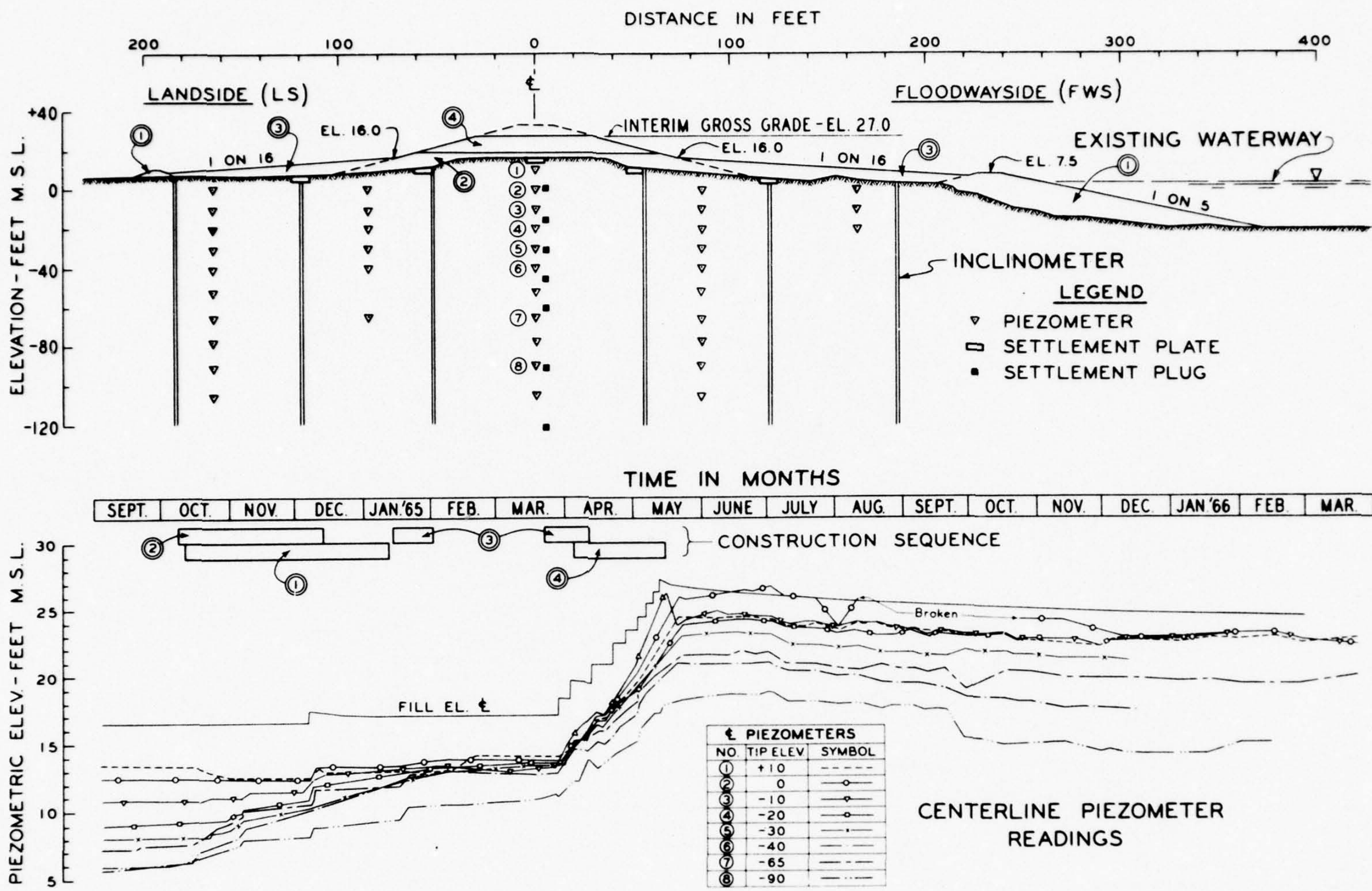
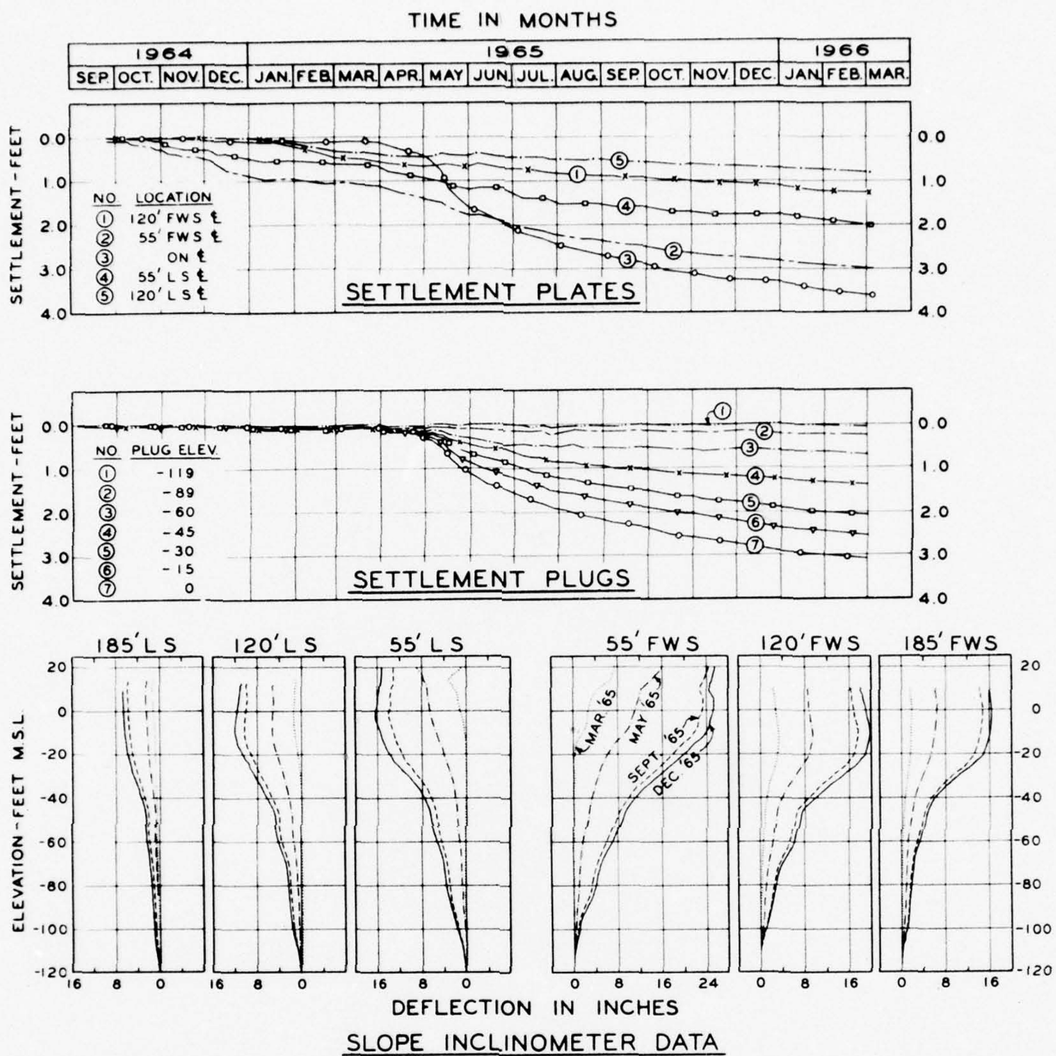


FIG. 2-2 TEST SECTION 2, TYPICAL SECTION, CONSTRUCTION SEQUENCE & PIEZOMETER DATA



NOTE: FOR SEQUENCE OF CONSTRUCTION SEE FIG. 2-2

FIG. 2-3 TEST SECTION 2, TYPICAL SETTLEMENT & MOVEMENT DATA

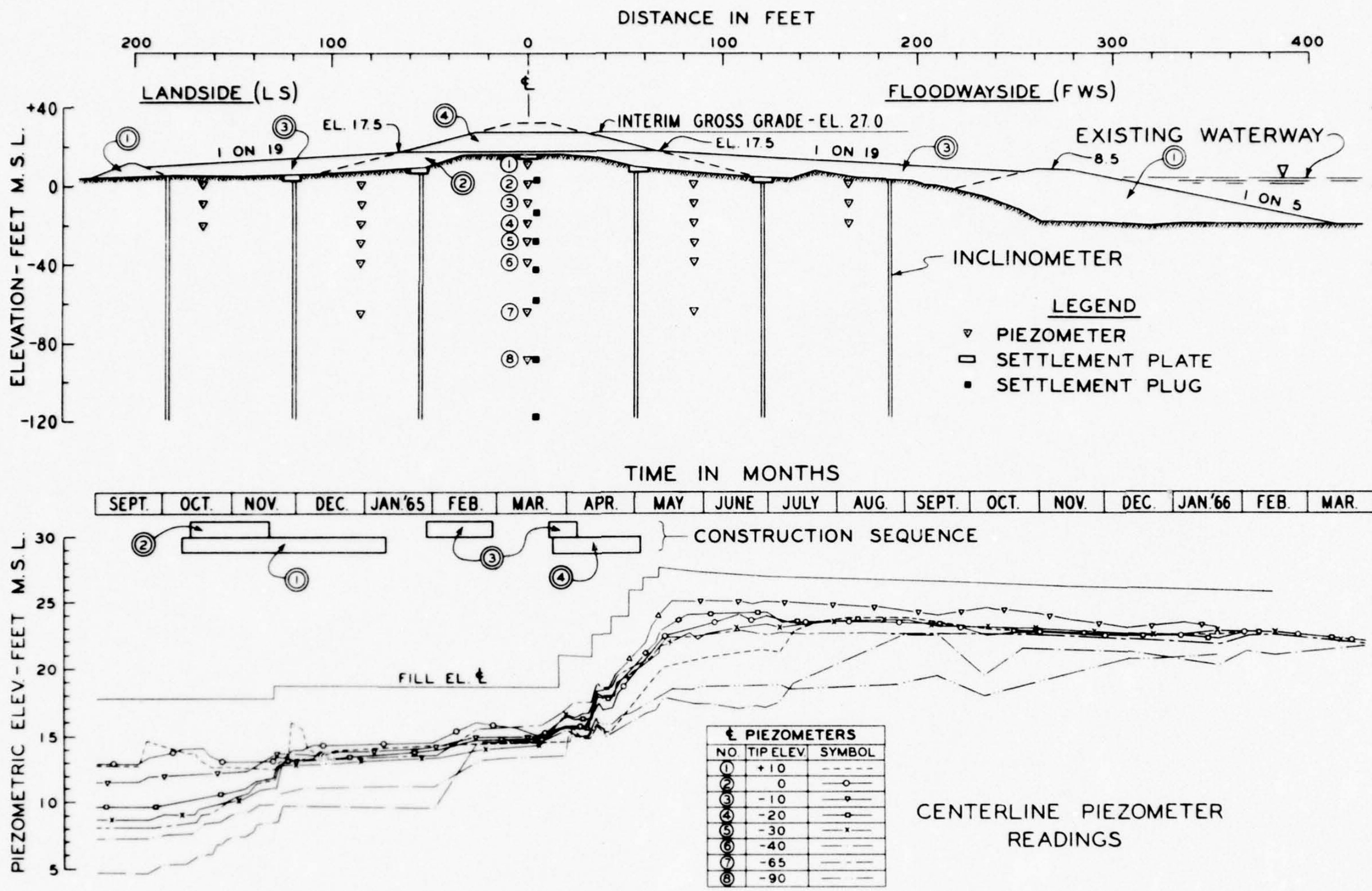
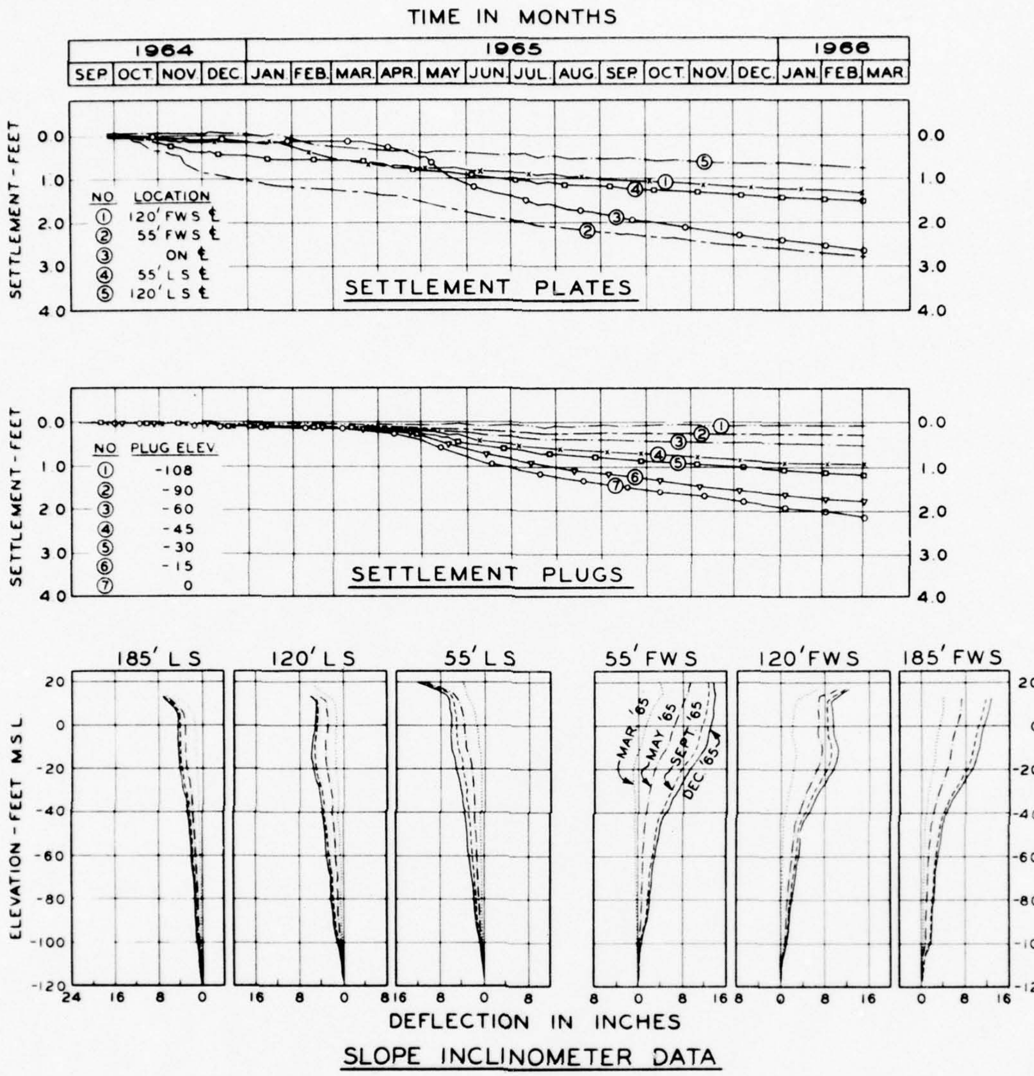


FIG. 2-4 TEST SECTION 3, TYPICAL SECTION, CONSTRUCTION SEQUENCE & PIEZOMETER DATA



NOTE: FOR SEQUENCE OF CONSTRUCTION
SEE FIG. 2-4

FIG. 2-5 TEST SECTION 3, TYPICAL SETTLEMENT & MOVEMENT DATA

3. GEOLOGY OF THE TEST SECTION AREA

The geology of the Atchafalaya Basin may usefully be traced from the last ice age at which time lowering of the sea level led to the formation of a vast valley as the streams which later would form the Mississippi River cut deep into the pleistocene deposits. With subsequent rise in sea level a thick bed of sand and gravel was deposited in this valley, after which a bed of clay was laid down. The clay is over 100 ft thick at the Test Section location.

During the period of clay deposition the Mississippi River occupied several routes through the valley. Each time natural levees and adjacent backswamp deposits were formed until the route was elevated to the point where a preferential route became available. The Atchafalaya Basin was formed by the joining of the natural levees of two of these routes, the Teche and the LaFourche. Behind these natural levees water was impounded to form a large lake in the southern part of the Atchafalaya Basin, the remains of which still exist as the Grand Lake.

About 1500 A.D. the Mississippi broke into the Atchafalaya Basin and joined with the Red River to form the Atchafalaya River. Since that time heavy sedimentation has occurred in the basin and the lake has been partially filled by deltaic deposits.

The geological sequence is described by Fisk et al (1952), who also detail the Test Section foundation clays as being backswamp deposits.

Krinitzsky and Smith (1969) investigated the geology of the backswamp deposits in the Atchafalaya Basin and divided them into three types on the basis of depositional environment--namely, well-drained swamp, poorly drained swamp, and lake deposits. They present typical properties for each deposit, such as the ranges of plasticity shown in Fig. 3-1. On the basis of these properties and after examining the boring profiles and classification test results, the virgin profile at the Test Section location probably consists of a layer of poorly drained

swamp deposit above El. -25, overlying lake and well-drained swamp deposits to approximately El. -120. Of interest is a layer of red peaty soil at about El. -25, which probably was deposited by the Red River after the formation of the Atchafalaya Basin but before the Mississippi River broke into the Basin.

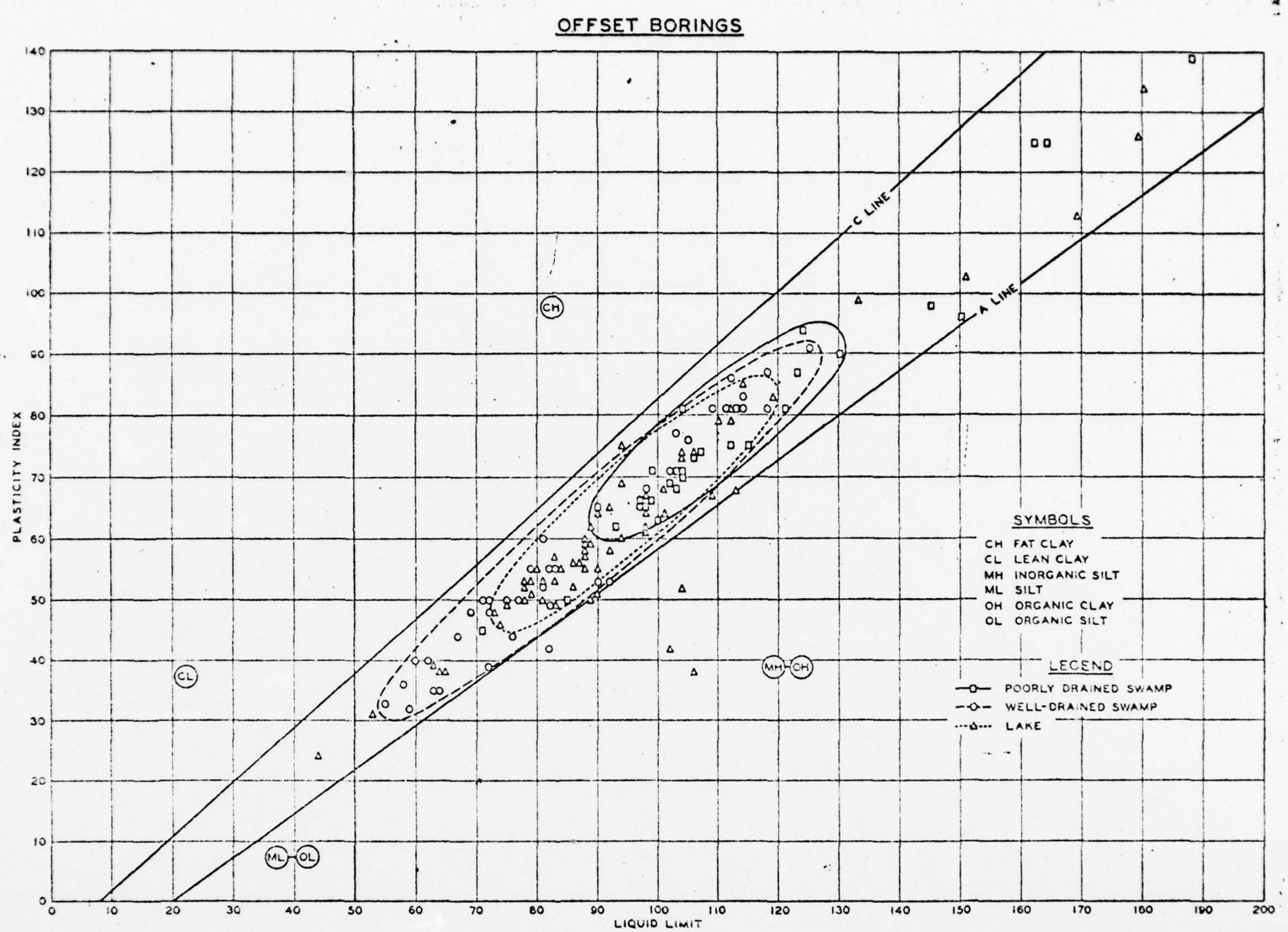


Fig. 3-1. Range in plasticity characteristics of Atchafalaya backswamp materials
Krinitzsky and Smith (1969)

4. STRESS HISTORY OF THE TEST SECTION FOUNDATION SOILS

4.1 DATA

Prior to design of the Test Sections, a comprehensive site investigation and testing program was performed. This consisted of a grid of borings with a series of classification tests, water content and density determinations, strength tests and oedometer tests. Borings were placed at six stations along Test Sections II and III, with borings being made at the center-line of the new levee and at 105 ft and 180 ft offsets to each side, giving a total of thirty borings in all. The resulting data, presented in the Interim Report (USCE, 1968), are the basis for this investigation of the stress history of the foundation soils.

Consideration of the construction history for the existing levees revealed that initially the entire area was level at approximately El. +2. In the 1930's the first levee in this area was constructed using material excavated from an adjacent borrow pit located floodway side of the levee. In later years this procedure was repeated to raise and enlarge the levee, the borrow pit becoming a segment of the Gulf Intracoastal Waterway. The initial soils profile throughout the area was reasonably constant as evidenced by the boring logs, the presence of a horizontal dark brown peat layer at about El.-25, and by analyses of soil test data for borings at different locations. However, the previous construction had caused major changes in the soil profile beneath the levee, as reflected in the centerline boring logs.

Since the Test Sections were constructed directly above the existing levees and since the existing levees were of generally constant section, it was expected that the soil profiles at constant offsets from the new levee centerline would be the same. On this basis all data for each individual offset were considered together, allowing data from six boreholes to be combined. Careful checks were made to verify this assumption, and it was found to be very acceptable.

Additional studies revealed that this procedure could be taken further, for it became clear that the profiles at the 105 ft and the 180 ft offsets were essentially unaffected by previous levee construction and that although there were minor differences in ground level, for most purposes the data from all these borings could be analysed together. The suitability of this assumption will be made apparent in later sections.

4.2 EFFECTIVE STRESS PROFILES

An individual profile for each offset was developed. The unit weight data for each offset were analysed and a total stress profile obtained. The preconstruction pore pressure profiles for each offset, as measured by the closed-system peizometers installed to monitor the Test Section performance, were then applied to the total stress profiles to give the effective stress profiles shown in Fig. 4-1. As can be seen, the profiles at 105 ft and 180 ft offsets are very similar. The profiles are shown as a band representing the range of measured pore pressures.

4.3 MAXIMUM PAST PRESSURE DATA

The next step in determining stress history was to determine the maximum past pressure, $\bar{\sigma}_{vm}$, profile for the foundation. For this purpose the $\bar{\sigma}_{vm}$ data for all four 105 ft and 180 ft offsets were analysed collectively, after first insuring the compatability of the four data sets by comparing the averages of the data for each individual offset.

Figure 4-2 shows the $\bar{\sigma}_{vm}$ data with an average line drawn through the values. This line was developed from average values for 10 ft layers and represents only one way of interpreting the data. The assumption of a linear increase in $\bar{\sigma}_{vm}$ with depth below El. -30 would seem, at first glance, to be just as acceptable an interpretation.

To try and reduce the scatter in these values, the original oedometer test data for all centerline and 180 ft offset borings were obtained and the consolidation curves replotted as strain versus effective stress using constant scales. Figure 4-3 shows a typical family of these test curves representing samples from the same boring but at increasing depths. Figure 4-4 presents the recalculated $\bar{\sigma}_{vm}$ data for 180 ft offsets and Fig. 4-5 shows the recalculated centerline data. Values from the M.I.T. tests on borings 300 ft to the landside are included in Fig. 4-4, since these borings would be representative of the virgin soil profile, as are the 180 ft offset borings. Effective stress profiles are also shown, the 105 ft to floodwayside profile being used for Fig. 4-4. As noted previously, this profile is very similar to that at the other 105 ft and 180 ft offsets.

These recalculated data do have less scatter than the Interim Report values plotted in Fig. 4-2, and Fig. 4-4 might be interpreted as showing certain areas of definite precompression in the virgin profile. At the centerline, the effective stress profile falls well above the maximum past pressures suggested by Fig. 4-4, and so it would be expected that the soil would be normally consolidated, except for the upper fill materials. Thus the $\bar{\sigma}_{vm}$ data of Fig. 4-5, which fall evenly about the vertical effective stress, seem appropriate and indicate that the average of the $\bar{\sigma}_{vm}$ values may be a good estimate of the in situ maximum past pressure.

Considering the average of the maximum past pressures from the 180 ft offsets, there is some indication of overconsolidated layers of soil at approximate elevations of -45 to -65 and -70 to -100. Further $\bar{\sigma}_{vm}$ data are available from borings at 105 ft offsets, but unfortunately the original test readings could not be obtained and so these data have not been recalculated. Figure 4-6 shows the 105 ft maximum past pressure data from the Interim

Report, but excluding data points from consolidation curves with poorly defined $\overline{\sigma}_{vm}$ values. A line based on average values for 10 ft layers is also shown. A similar average line for the recalculated 180 ft data is presented in Fig. 4-7. Figure 4-8 compares these two lines and the good agreement seems to indicate that although the scatter in the 105 ft data is much larger than with the recalculated 180 ft data, the average trend is the same in both cases. Thus the recalculated data at 180 ft offsets were considered to give a suitable indication of the $\overline{\sigma}_{vm}$ profile for the 105 ft and 180 ft offsets and indeed for the entire area except where previous levee construction had occurred. The data defined the same trends and with less scatter than the original data shown in Fig. 4-2.

Figure 4-9 shows the recalculated data within approximate limits of scatter and Fig. 4-10 shows the same data with an average line drawn through the data. These indicate definite areas of overconsolidation.

It should be emphasised that so far this interpretation of the $\overline{\sigma}_{vm}$ profile is very tentative. Additional indications of these overconsolidated layers are needed before their presence can be fully accepted.

4.4 EVIDENCE OF OVERCONSOLIDATED LAYERS IN THE TEST SECTION FOUNDATION

In addition to the $\overline{\sigma}_{vm}$ data discussed above, indication of overconsolidated layers is contained in the field vane strength data, the natural water content profiles, and the unit weight data.

4.4.1 Field Vane Strength Data

For each 105 ft and 180 ft offset, the unconfined test data and the field vane strength data for all six borings were averaged over 5 ft layers. The resulting average strength profiles are plotted in Fig. 4-11. The field vane data reflect the similarity of profiles at all four offsets and shows large increases in strength occurring at about El. -45 and -75. These increases are consistent with the zones of overconsolidation suggested by the $\bar{\sigma}_{vm}$ data. The unconfined test strength data do not show these large increases in strength. Sample disturbance effects are probably the reason for this (see Ladd, 1971).

4.4.2 Natural Water Content Profiles

The water content profiles for all borings at each offset except the centerline were plotted together and the bands enclosing these profiles are plotted in Fig. 4-12. Average lines were then drawn through each band and the four average lines so obtained plotted together in Fig. 4-13. The resulting data indicate lower water content in the elevation ranges -45 to -60 and -70 to -90. This is consistent with overconsolidated layers at these elevations.

Examination of the Interim Report, however, yields evidence that areas of low water content are accompanied by large reductions in liquid limit and small reductions in plastic limit, such that the liquidity index in these regions may not be lower than at neighboring elevations (for example, see boring 86). Thus the reduction in water content could also be interpreted as resulting from layering in the foundation, indicated by variation in the Atterberg limits.

However, reference to Fig. 3-1 allows an interpretation

of these data which is still consistent with the concept of overconsolidated layers. It is shown that well-drained swamp deposits have lower plasticities than the other backswamp deposits. A well-drained swamp would be associated with considerable weathering and desiccation. This would also be the mechanism by which overconsolidated layers would have occurred in the ground--namely, that they were surface drying crusts when the ground surface was at these elevations. Thus desiccation could have reduced the water contents and also caused reduction in the Atterberg limits, in which case these two tendencies would both be associated with overconsolidated layers.

4.4.3 Total Unit Weight Data

Variation in total unit weight with depth is closely correlated to variation in water content, so the same trends observed with water contents should occur with unit weights. Figure 4-14 shows a band which encloses the average unit weight profile for each of the 105 ft and 180 ft offsets. Increases in unit weight occur between elevations -45 and -60 and -70 and -90. This is consistent with the decreases in water content described above and with the probable presence of overconsolidated layers.

The averaging processes involved in developing Fig. 4-14 tend to give smooth curves instead of the sharp increases in unit weight which would be expected with drying crusts deep in the foundation.

4.5 STRESS HISTORY--CONCLUSION

The $\bar{\sigma}_{vm}$ data suggest that layers of overconsolidated soil exist in the foundation at approximate elevations of -45 to -60 and -70 to -100. Examination of shear strength data, natural water content trends, and unit weight profiles reveals that the variations with depth of each of these can be explained

in a way consistent with the presence of overconsolidated layers at these elevations. While the interpretation of each individual set of data as indicating the presence of overconsolidated layers is open to question, when considered collectively it is felt that the presence of these layers is adequately proven. Geologically it is very possible that they could exist, being drying crusts formed in the backswamp deposits.

Thus the $\bar{\sigma}_{vm}$ profile, where not affected by previous levee construction, is as shown in Fig. 4-10. This profile applies to the 105 ft and 180 ft offsets and to the foundation material beneath the floodway side borrow pit. Using the effective stress profiles of Fig. 4-1 and calculating the profile beneath the floodway side borrow pit using unit weights for the 105 ft and 180 ft offsets, values of effective vertical stress and the overconsolidation ratio ($OCR = \bar{\sigma}_{vm}/\bar{\sigma}_{vo}$) for different elevations at each of these locations can be obtained. Values for the 105 ft and 180 ft offsets to the floodway side of the levee centerline are shown in Tables 4-1 and 4-2, and values beneath the floodway side borrow pit are shown in Table 4-3. Beneath the centerline the soil is normally consolidated except for a drying or compaction crust as shown in Table 4-4.

STRESS HISTORY: 105 FT FS OF CENTERLINE

El. (ft)	$\bar{\sigma}_{vo}$ (psf)	$\bar{\sigma}_{vm}$ (psf)	OCR = $\bar{\sigma}_{vm}$ / $\bar{\sigma}_{vc}$
+ 6.4	0		
+ 5	100	600	6.00
0	300	1200	4.00
- 5	520	1000	1.92
-10	620	700	1.13
-15	680	680	1.00
-20	730	730	1.00
-25	840	840	1.00
-30	950	950	1.00
-35	1100	1100	1.00
-40	1270	1270	1.00
-45	1400	2300	1.64
-50	1570	2200	1.40
-55	1770	2100	1.19
-60	1970	2100	1.07
-65	2150	2150	1.00
-70	2330	4000	1.72
-75	2550	3850	1.51
-80	2750	3800	1.38
-90	3170	3800	1.20
-100	3600	3800	1.05

Table 4-1

STRESS HISTORY: 180 FT FS OF CENTERLINE

El. (ft)	$\bar{\sigma}_{vo}$ (psf)	$\bar{\sigma}_{vm}$ (psf)	OCR
+ 3.2	0		
0	120	1200	10.00
- 5	375	1000	2.66
-10	460	700	1.52
-15	525	525	1.00
-20	590	590	1.00
-25	700	700	1.00
-30	820	820	1.00
-35	1000	1000	1.00
-40	1170	1170	1.00
-45	1400	2300	1.64
-50	1570	2200	1.40
-55	1770	2100	1.19
-60	1970	2100	1.07
-65	2150	2150	1.00
-70	2330	4000	1.72
-75	2550	3850	1.51
-80	2750	3800	1.38
-90	3170	3800	1.20
-100	3600	3800	1.05

Table 4-2

STRESS HISTORY: BENEATH THE FLOODWAY SIDE BORROW PIT

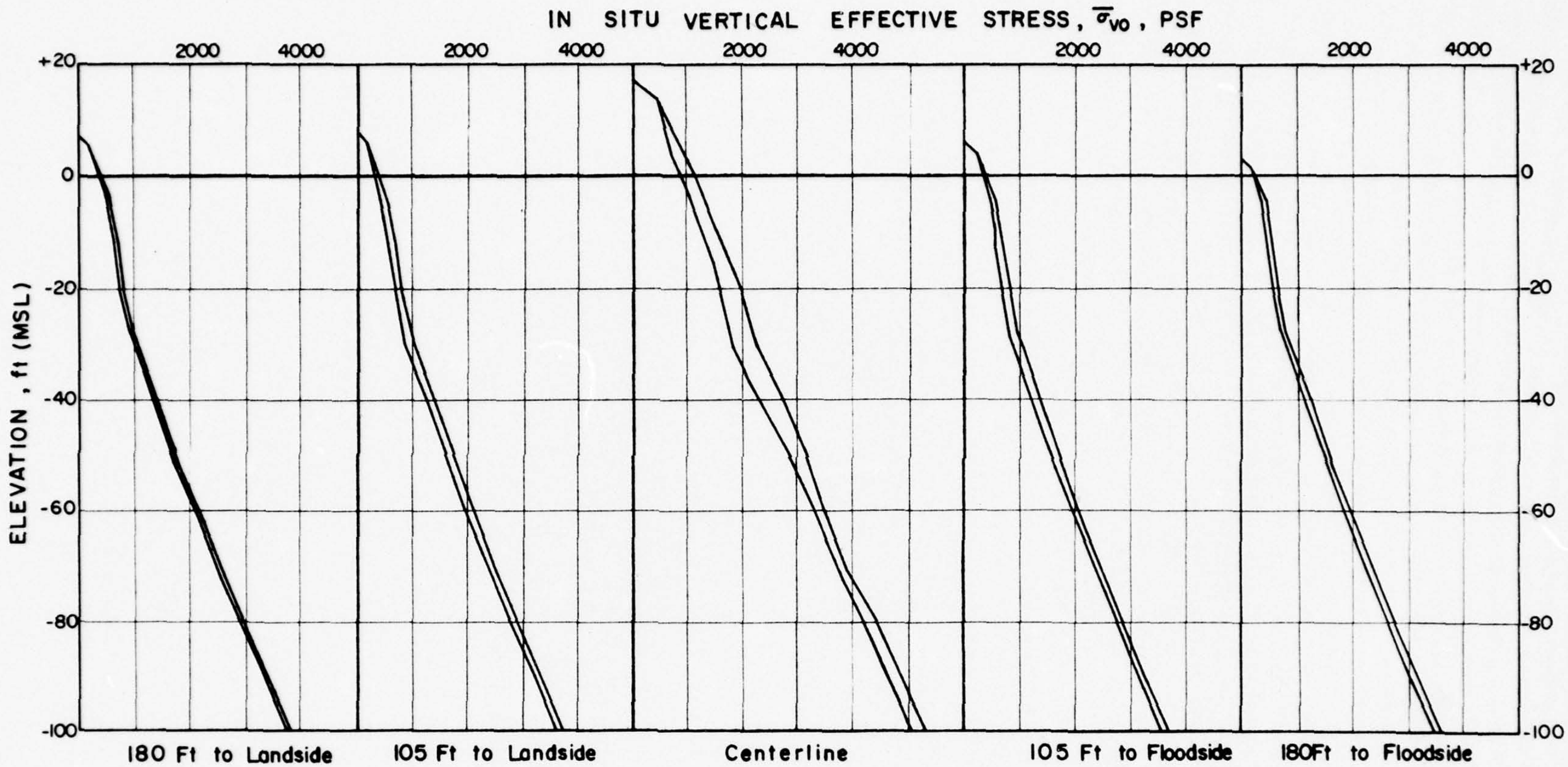
El. (ft)	$\overline{\sigma}_{vo}$ (psf)	$\overline{\sigma}_{vm}$ (psf)	OCR
-20	0	600	
-25	100	700	7.0
-30	220	800	3.6
-35	400	1000	2.5
-40	570	1150	2.0
-45	800	2300	2.9
-50	970	2200	2.3
-55	1170	2100	1.8
-60	1370	2100	1.5
-65	1550	2150	1.4
-70	1730	4000	2.3
-75	1950	3850	2.0
-80	2150	3800	1.8
-90	2570	3800	1.5
-100	3000	3800	1.3

Table 4-3

STRESS HISTORY: CENTERLINE

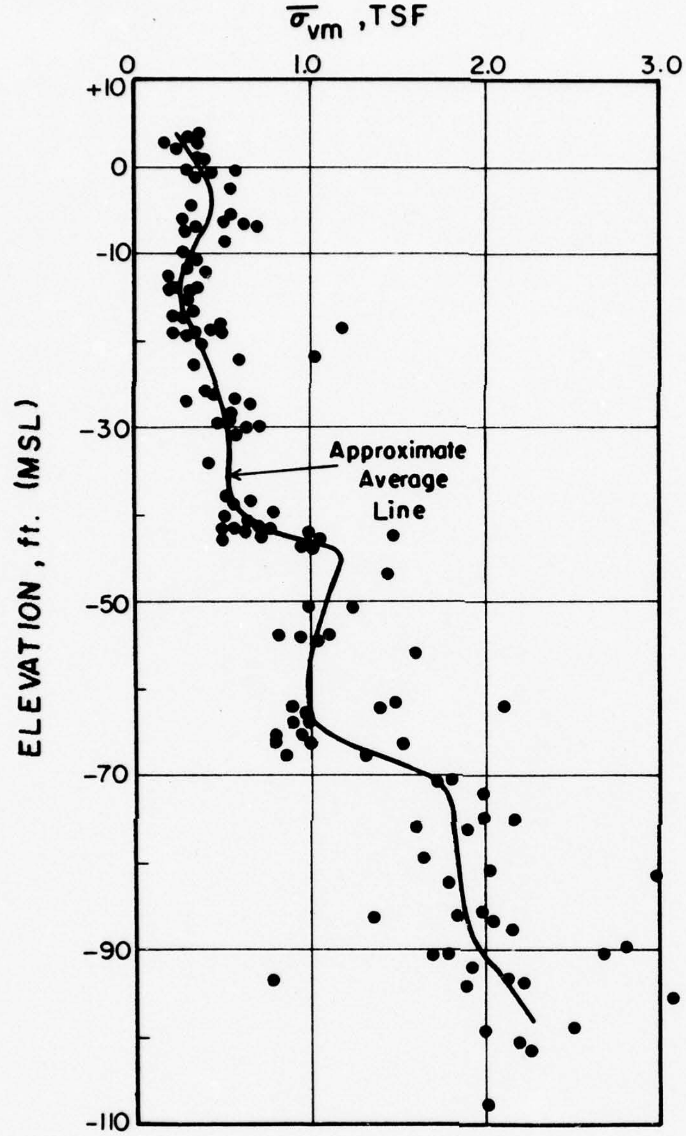
El. (ft)	$\bar{\sigma}_{vo}$ (psf)	$\bar{\sigma}_{vm}$ (psf)	OCR
+ 17.5	0	2200	
+ 15	265	2000	7.55
+ 5	780	1750	2.25
0	1000	1700	1.70
- 5	1170	1700	1.45
- 10	1380	1700	1.23
- 15	1610	1700	1.05
- 20	1760	1760	1.00
- 25	1880	1880	1.00
- 30	2030	2030	1.00
- 35	2250	2250	1.00
- 40	2500	2500	1.00
- 45	2740	2740	1.00
- 50	2980	2980	1.00
- 60	3390	3390	1.00
- 70	3800	3800	1.00
- 80	4310	4310	1.00
- 90	4780	4780	1.00
-100	5210	5210	1.00

Table 4-4



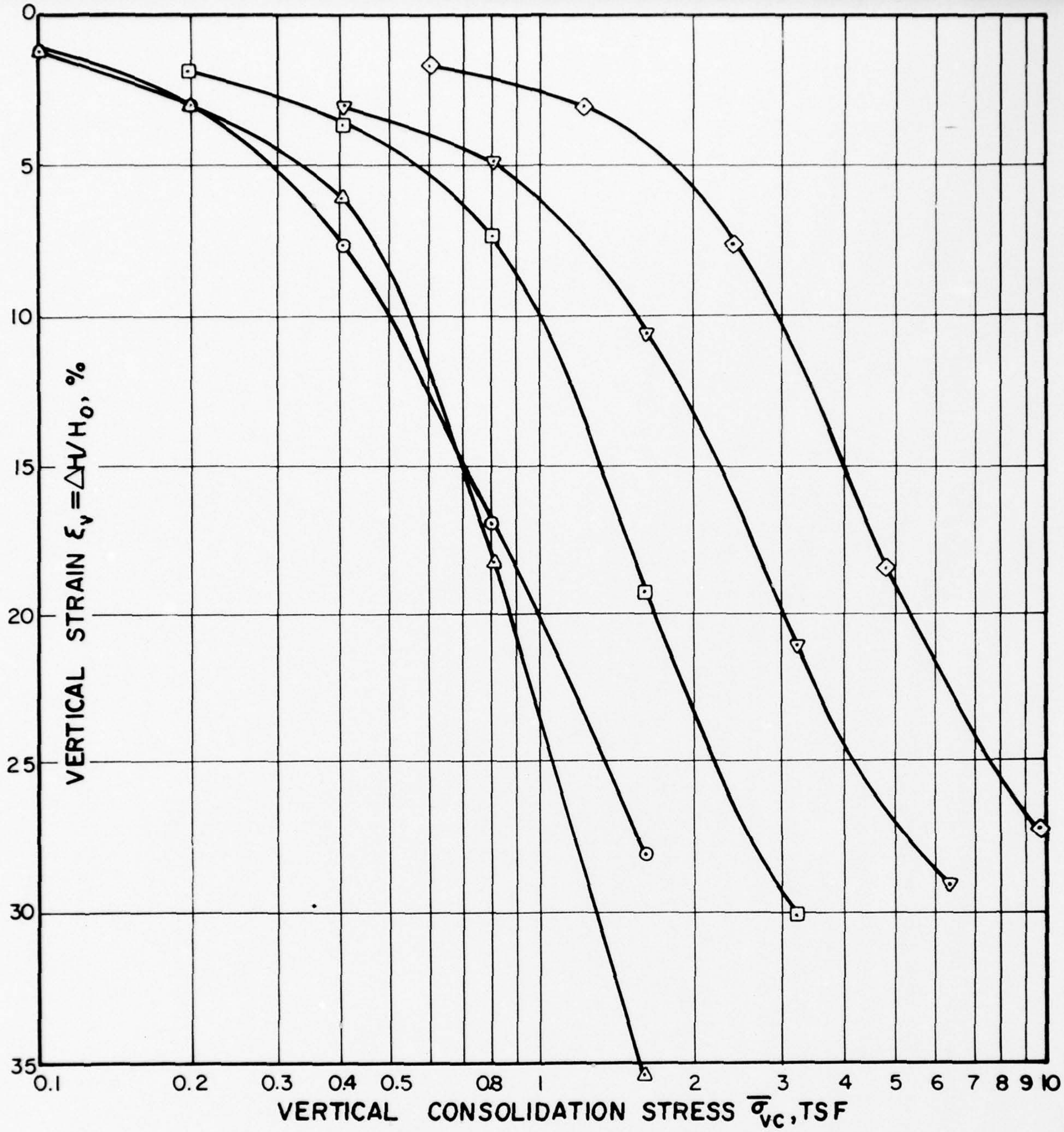
PRE – CONSTRUCTION EFFECTIVE VERTICAL STRESS PROFILES, TEST SECTIONS II & III

FIGURE 4 – I



MAXIMUM PAST PRESSURE DATA: OFFSETS 105 FT. & 180 FT. TO LS & FS (ALL DATA FROM INTERIM REPORT, USCE 1968)

FIGURE 4-2

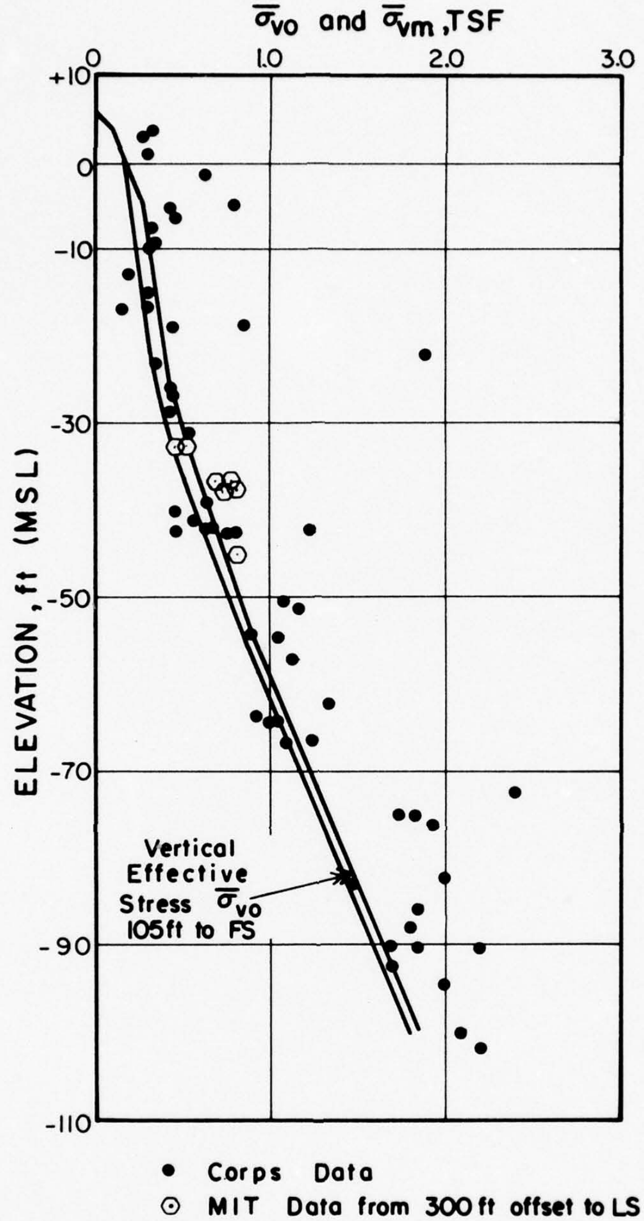


Symbol	W_N or e_0	El., ft.	$\bar{\sigma}_{vo}$ p.s.f.	$\bar{\sigma}_{vm}$ p.s.f.	REMARKS
⊙	120	-9.2	675	730	
△	351	-6.5	600	940	
⊡	85.5	-18.8	750	1700	
▽	62	-42.6	1450	2450	
◇	63.5	-90.4	3400	4400	

Boring 87-UE Sta 1396+50, 180 ft LS

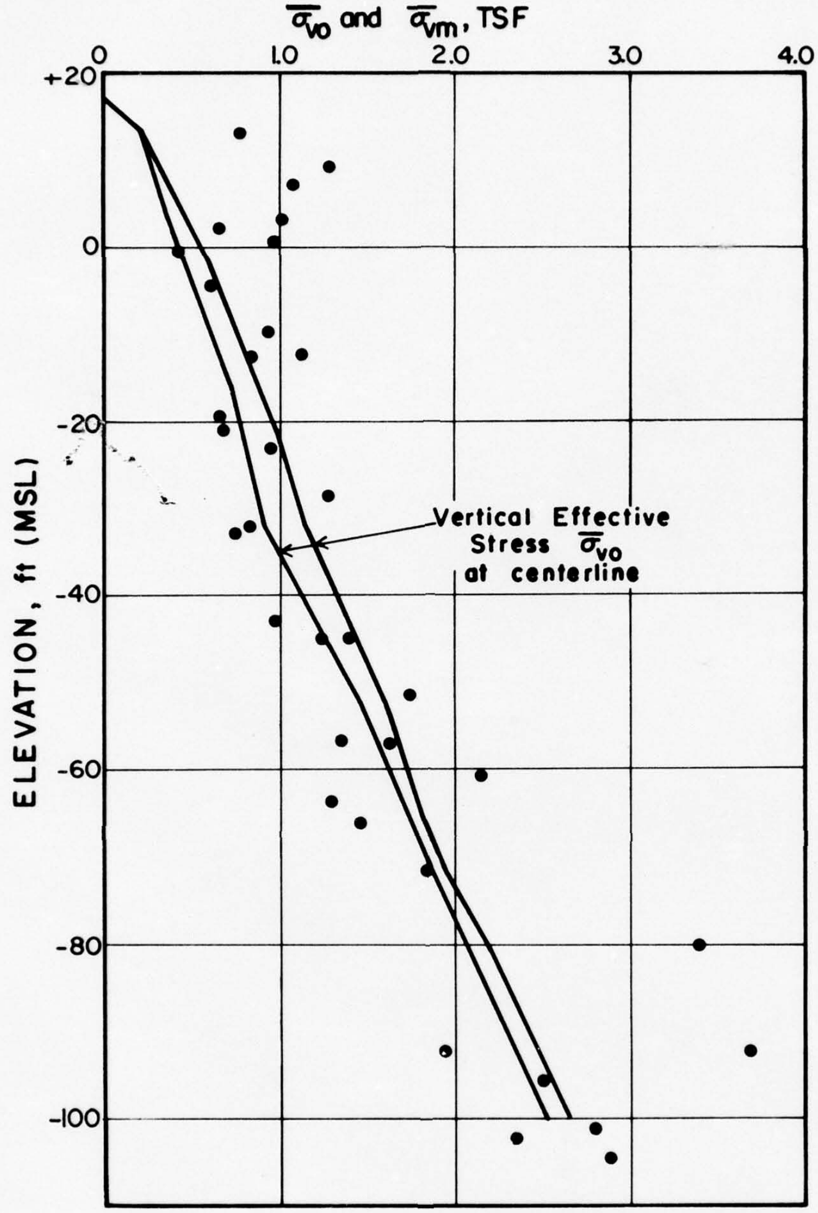
COMPRESSION CURVES FOR EABPL FOUNDATION CLAYS

FIGURE 4-3



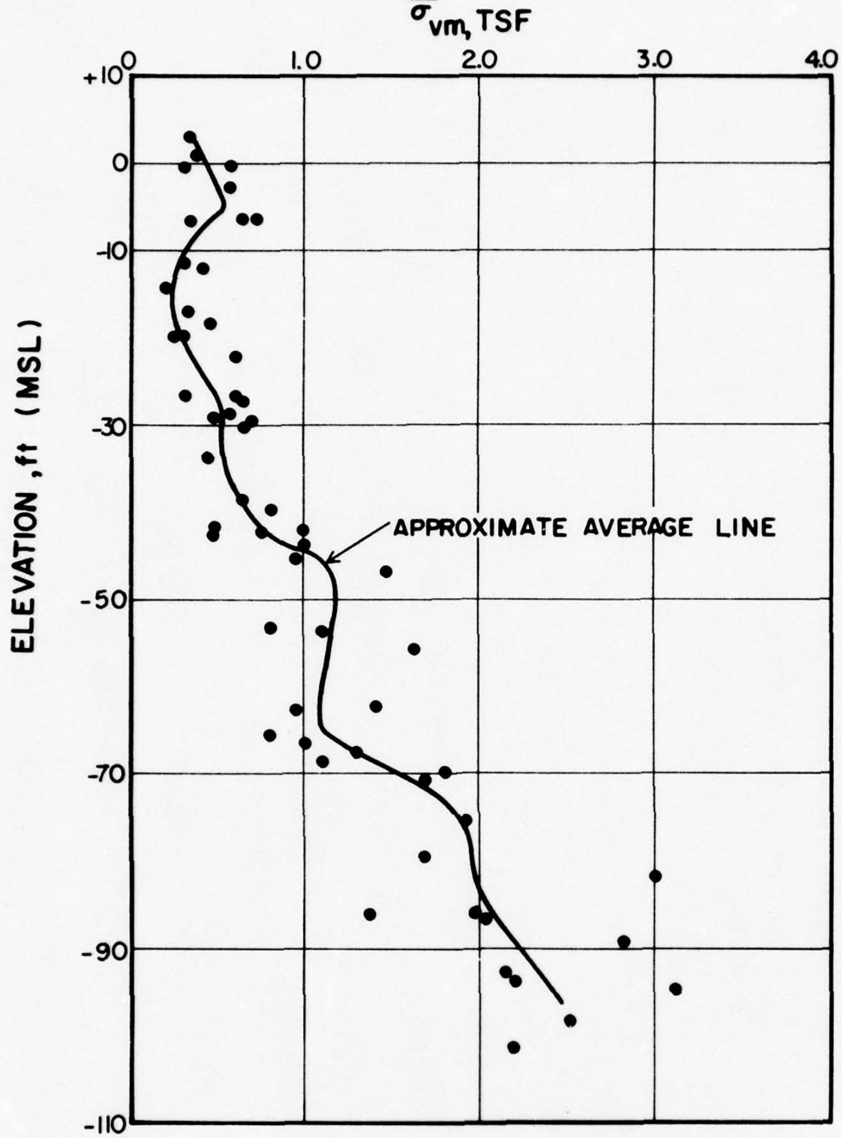
RECALCULATED MAXIMUM PAST PRESSURE DATA, SECTIONS II AND III: 180 FT OFFSETS TO LS AND FS

FIGURE 4-4



RECALCULATED MAXIMUM PAST PRESSURE DATA, SECTIONS II AND III CENTERLINE

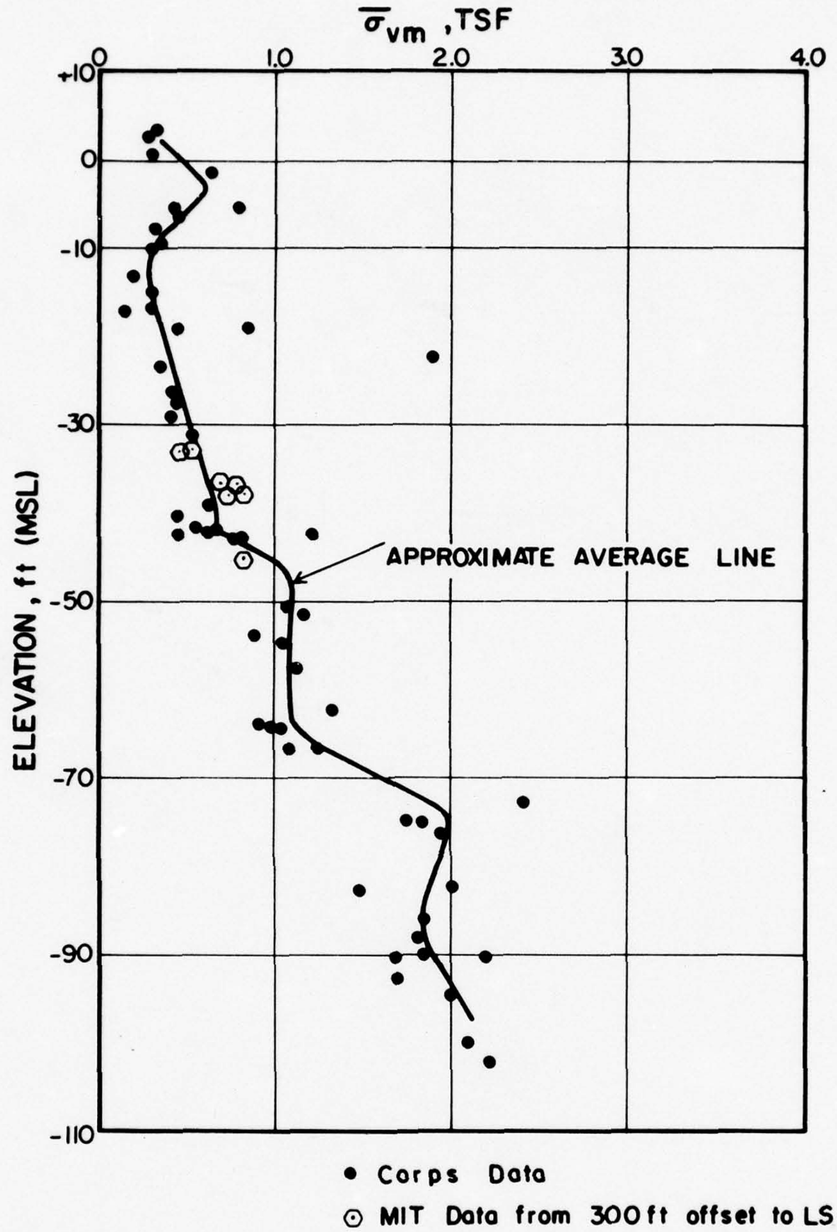
FIGURE 4-5



Data from Interim Report with values from poor e vs log $\overline{\sigma}_{vc}$ curves not included

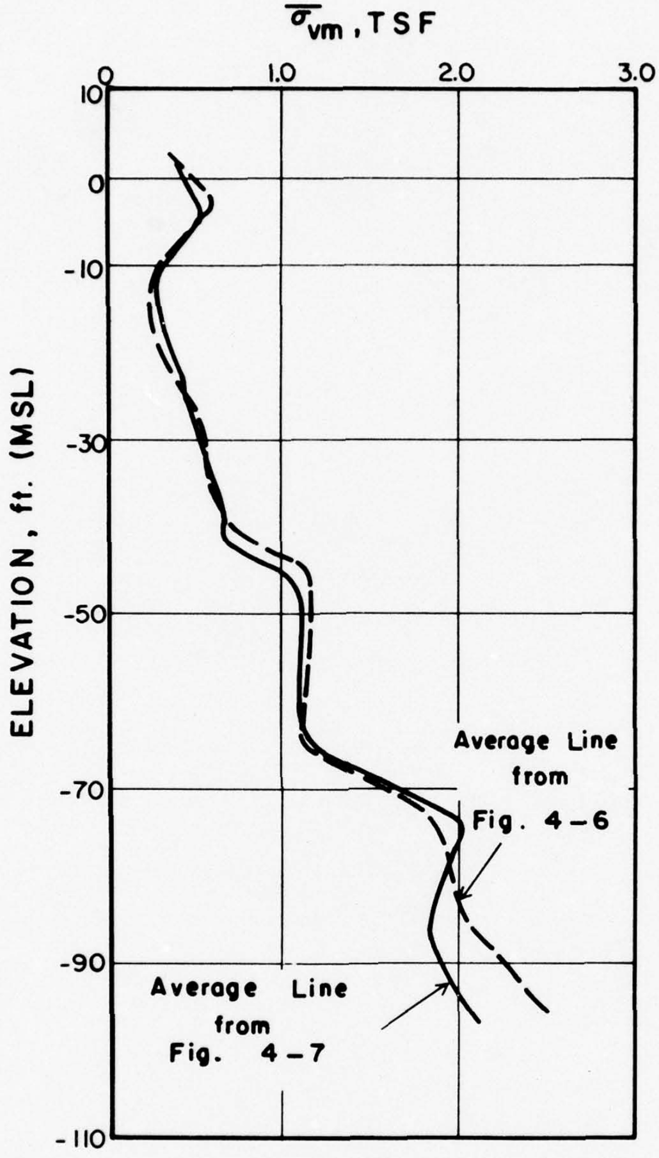
MAXIMUM PAST PRESSURE DATA, SECTION II AND III AT 105 FT TO LS AND FS

FIGURE 4-6



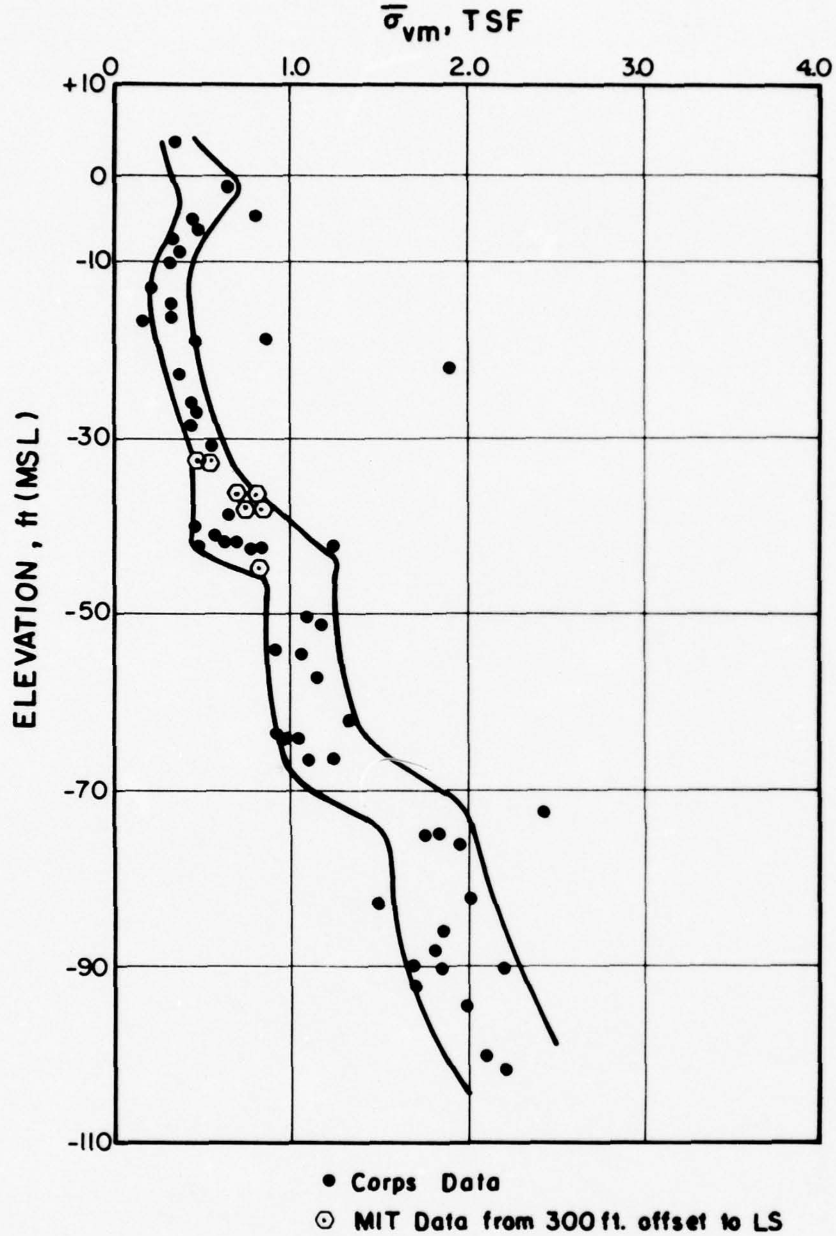
RECALCULATED MAXIMUM PAST PRESSURE DATA, SECTIONS II AND III, 180 FT. OFFSETS TO LS AND FS

FIGURE 4-7



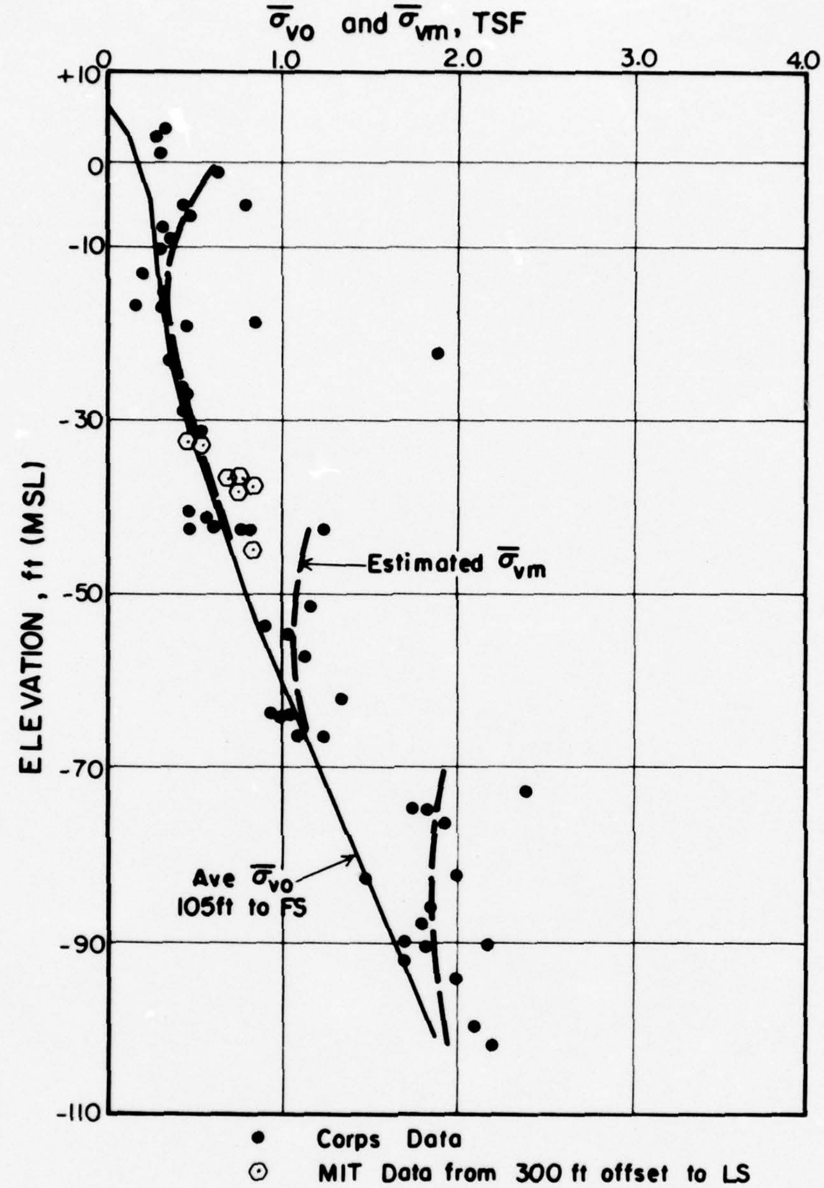
COMPARISON OF AVERAGE LINES FROM FIGURES 4-6 AND 4-7

FIGURE 4-8



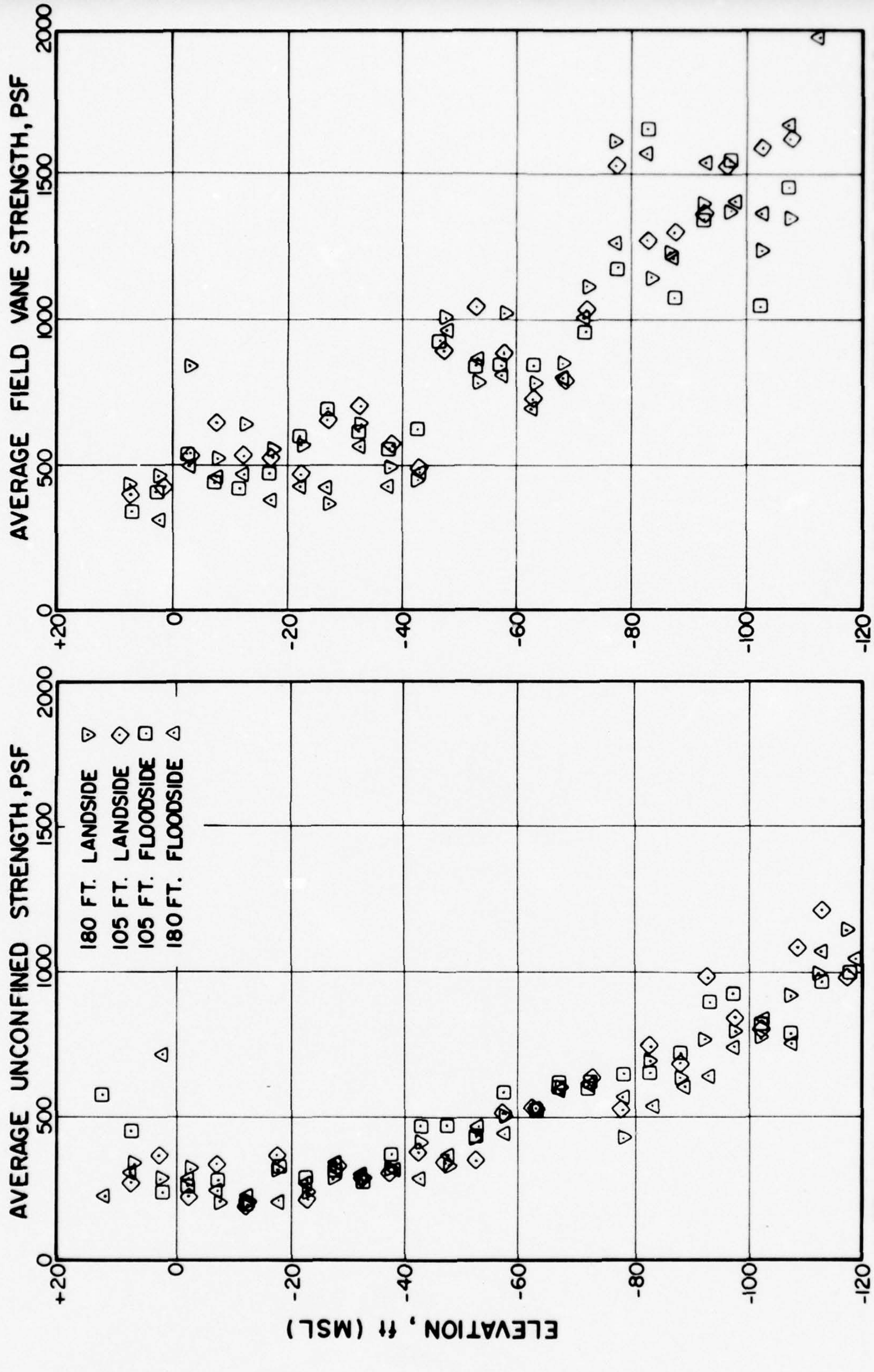
RECALCULATED MAXIMUM PAST PRESSURE DATA,
SECTIONS II AND III, 180 FT OFFSET TO LS AND FS

FIGURE 4-9



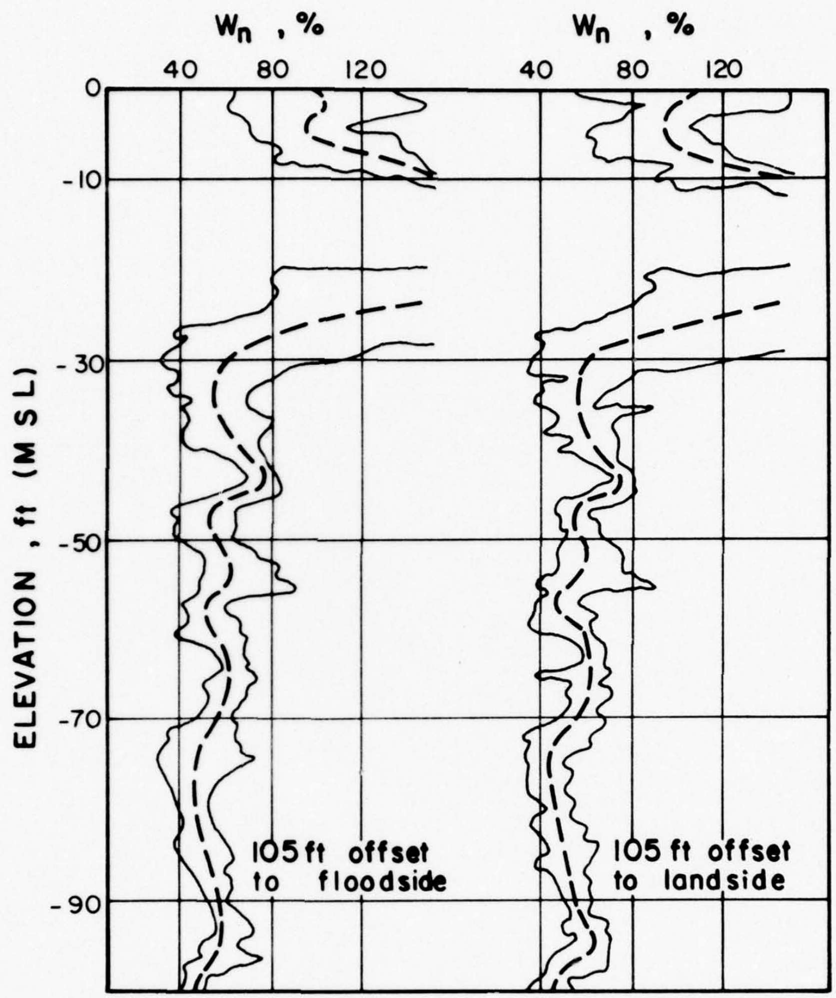
ESTIMATED MAXIMUM PAST PRESSURE, SECTIONS II AND III, 180 FT OFFSETS TO LS AND FS

FIGURE 4-10



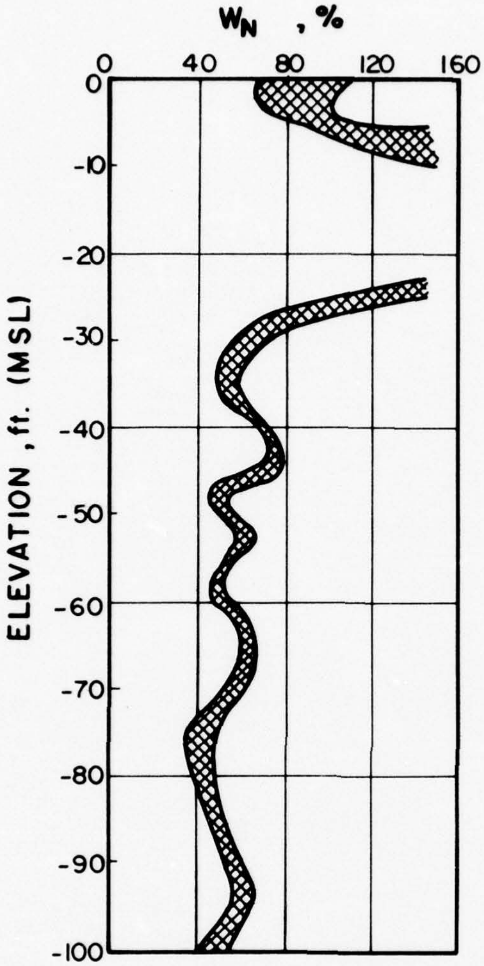
AVERAGE UNCONFINED AND FIELD VANE STRENGTH DATA FROM TEST SECTIONS II & III

FIGURE 4-11



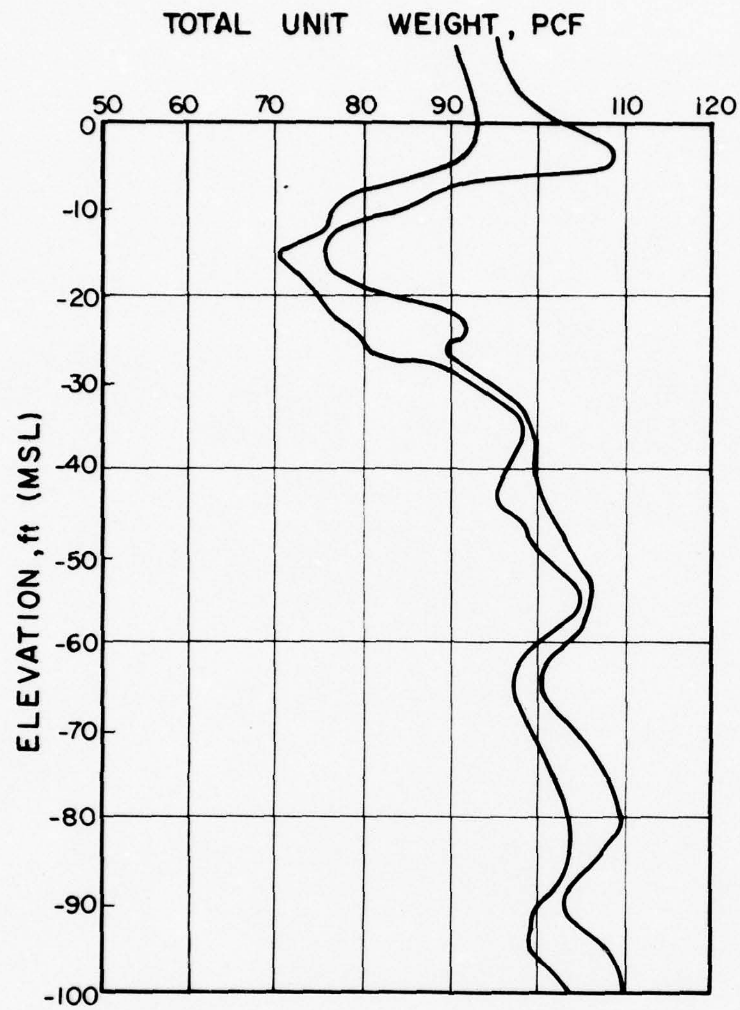
NATURAL WATER CONTENT VERSUS ELEVATION, SECTIONS II & III, 105 FT OFFSETS TO LS AND FS

FIGURE 4-12



AVERAGE NATURAL WATER CONTENT
PROFILES FOR 105 FT & 180 FT OFFSETS
FOR SECTIONS II & III

FIGURE 4-13



RANGE OF AVERAGE TOTAL UNIT WEIGHT VALUES, SECTIONS II & III, AT OFFSETS OF 105 FT. & 180 FT. TO EACH SIDE OF CENTERLINE

FIGURE 4-14

5. EVALUATION OF SOIL PROPERTIES AND SELECTION OF SOIL PARAMETERS

5.1 INTRODUCTION

The finite element analysis of the embankment required knowledge of the following soil properties:

(1) Undrained shear strength, s_u, and its variation with stress system and OCR

(2) Undrained Youngs modulus, E_u, and its variation with stress level, stress system, and OCR

(3) Lateral earth pressure coefficient at rest, K_o, and its variation with OCR.

In evaluating these properties, extensive use was made of the normalized soil parameters concept by which strength or modulus is normalized against a soil property such as vertical effective stress or undrained strength, and the resulting parameters are applied to the entire clay profile. Use of such parameters was justified by data from the M.I.T. soil testing program (Ladd, et al, 1972) for all foundation soils except the layer of very high water content material above El. -25. Samples of this material were not tested at M.I.T., so no positive proof exists that normalized parameters obtained for the lower deposits are appropriate for this material, except that the Corps of Engineers R testing program did not indicate any significant lowering in normalized undrained strength for that layer. Also the strength and modulus parameters used for the layer were quite low, and it is unlikely that they would be significantly in error. However, their use remains an assumption in the finite element model.

Extensive analysis of M.I.T. test data contained in the report on soil properties (Ladd et al, 1972) resulted in the evaluations presented in the following subsections.

5.2 UNDRAINED SHEAR STRENGTH

The M.I.T. testing program showed the undrained shear strength measured in consolidated-undrained (CU) triaxial compression tests to be very dependent on the strain rate at which the tests were performed. However, extrapolating results down to the very slow strain rates which apply with the field construction loading gives exceedingly low strengths and is totally impractical. Since strain rate effects occur with most clays and the Atchafalaya strain rate effects were not excessive for clays of high plasticity, experience and empirical data are needed to aid in strength selection.

Undrained laboratory strength data have been used for many years to give quantitative strength values which are used directly for design, generally with reasonable success. Research into the strength characteristics of clays has shown the subject to be extremely complex and indicates that the suitability of values obtained from standard laboratory testing programs is largely the result of compensating errors. The important conclusion here is that using standard strain rates has usually given strength values accurate enough for use with normal factors of safety. Thus there is some basis for using the strength results from normal testing strain rates and then reviewing the finite element analysis results with full regard to the possible consequences of this assumption. This approach has been used.

The alternative procedure would require a thorough investigation of how strain rate effects measured in the laboratory compare with strain rate effects in situ, which would be an entire study in itself.

Based on the above reasoning, the following strength parameters were selected for the finite element analyses of undrained deformations.

In Situ Stress System	Undrained Strength Ratio, $s_u / \bar{\sigma}_{vo}$
1. Plane Strain Active ($\sigma_{1f} = \sigma_{vf}$)	0.26
2. Direct-Simple Shear (Horizontal failure plane)	0.24
3. Plane Strain Passive ($\sigma_{1f} = \sigma_{hf}$)	0.22

where : σ_{1f} = major principal stress at failure

σ_{vf}, σ_{hf} = vertical, horizontal stress at failure

The effects of overconsolidation on $s_u / \bar{\sigma}_{vo}$ were well defined by the $\overline{CK_oU}$ direct-simple shear testing program. Figure 5-1 presents the reduction in the direct-simple shear s_u with increasing OCR. It was assumed that the plane strain values of $s_u / \bar{\sigma}_{vo}$ followed the same relationship.

5.3 UNDRAINED YOUNGS MODULUS, E_u

The finite element analysis used a bilinear stress versus strain representation for the soil. For selection of parameters, the undrained modulus, E_u, was normalized by the undrained strength, s_u, and the variation of E_u / s_u with stress level, stress system, and OCR was investigated.

The modulus is important in two ways. Variation of the absolute level of modulus used causes an inverse variation in the magnitude of deformations predicted by the finite element analysis. Increasing the moduli throughout the foundation soil by a factor of two will result in deformations reduced by a factor of two, but with the same shape. There is no good way

of determining the correct absolute value of modulus from laboratory tests (See D'Appolonia, Poulos, and Ladd, 1971). However, Fig. 5-2 presents data comparing modulus values for several normally consolidated soils, from which it is apparent that the Atchafalaya clay has much lower values of E_u / s_u than other soils for which experience is available. This knowledge, plus the empirical observation that $\overline{CK_oU}$ direct-simple shear tests tend to give E_u / s_u values which are generally appropriate for field conditions, allowed the absolute level of modulus to be selected to give deformations that should be the correct order of magnitude.

The second way in which modulus is important is that its variation with stress level, stress system, and OCR throughout the foundation has a very significant effect on the shape of deformations obtained by the analyses. Since it is the shape of the deformations and particularly the highly stressed zone between El. -20 and -40 that are the most significant aspects of measured behavior, appropriate representation of the variation in modulus throughout the foundation is extremely important. After analysis of the modulus values from the testing program, the following general variations in E_u / s_u with stress system and stress level were indicated.

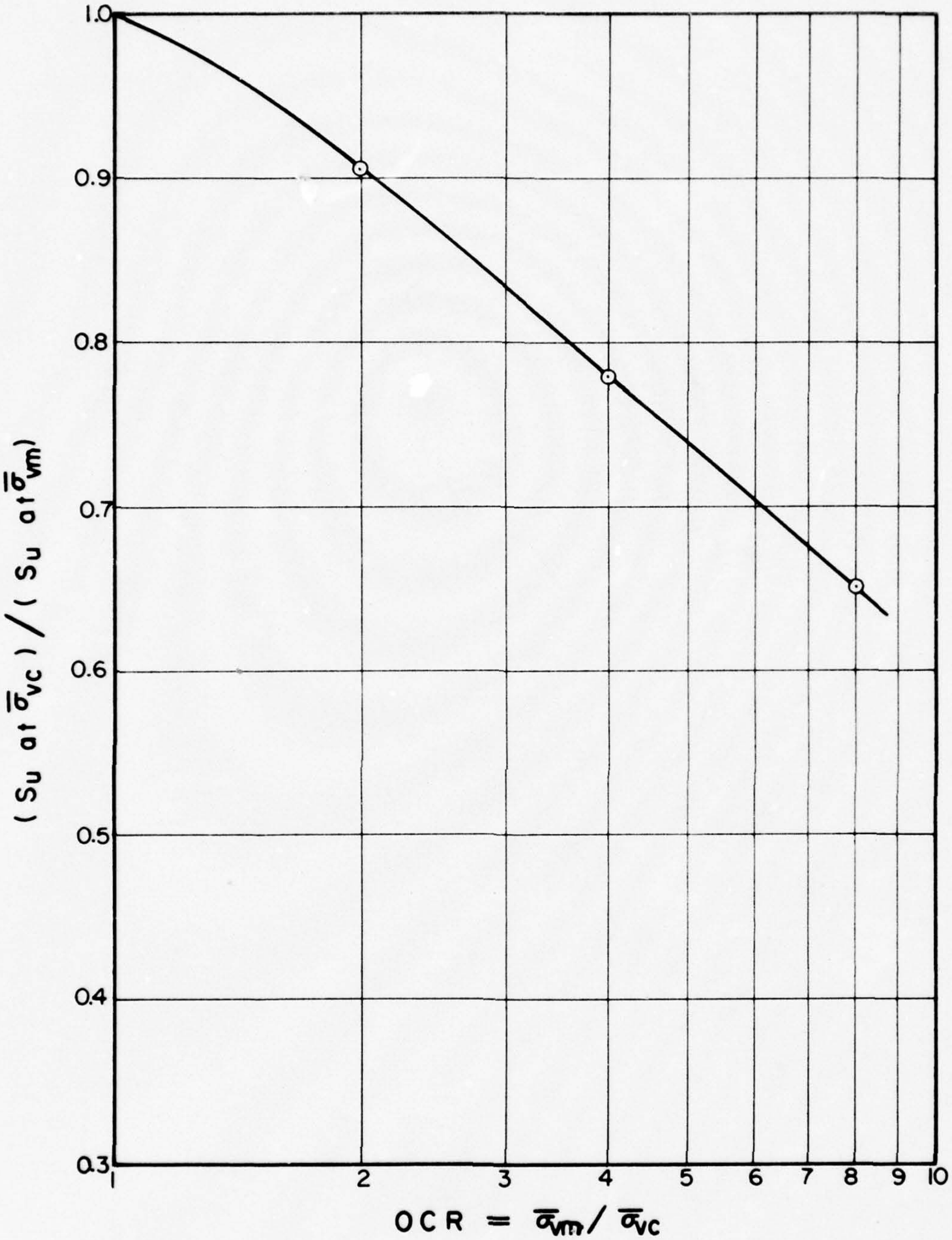
Type of $\overline{CK_oU}$ Test	E_u / s_u FS = 5	FS = 2	FS = 1.25
1. Triaxial Compression	--	300	100
2. Direct-Simple Shear	400	150	50
3. Triaxial Extension	320	110	50

These data were developed from a relatively limited number of tests and are subject to large variation. However, they do serve to indicate large changes in modulus with stress level and stress system. The trends indicated were used in the selection of moduli for the foundation soils.

Variation of E_u / s_u with OCR was shown by the $\overline{CK_oU}$ direct-simple shear data (Ladd et al, 1972). With OCR's of 2 to 4 a reduction in E_u / s_u of about 50 percent occurs at a factor of safety of 5, reducing to almost no change at a factor of safety of 1.25. This trend was also used in the selection of moduli.

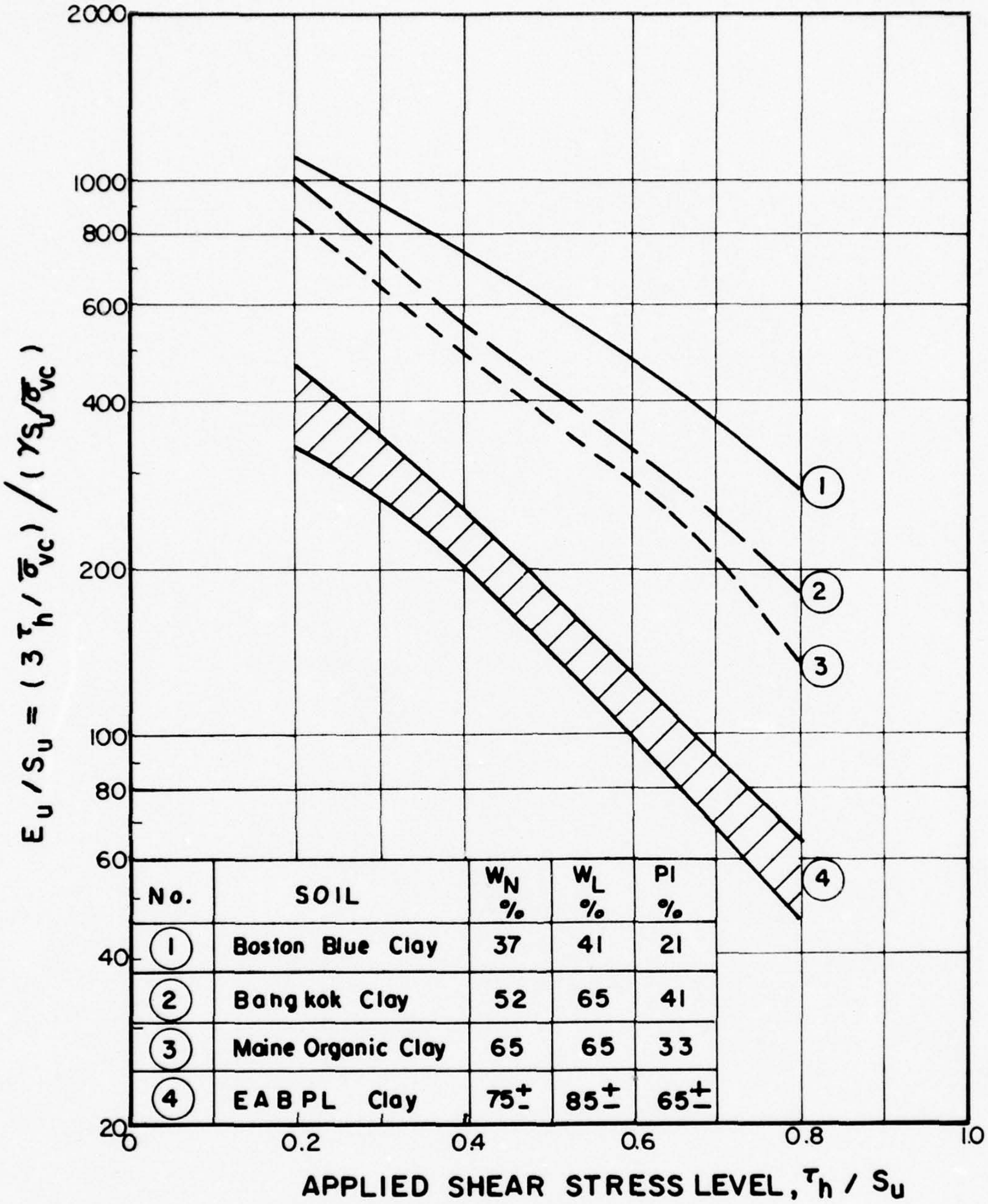
5.4 LATERAL EARTH PRESSURE AT REST, K_o

Brooker and Ireland (1965) give very complete data on K_o and its variation with OCR for a range of soils. Plane strain active tests indicated a K_o of about 0.67 for normally consolidated samples. This is in agreement with Brooker and Ireland's value for soils of this plasticity, and so their data were used to extrapolate K_o to other OCR's.



DECREASE IN S_u WITH REBOUND FROM $\overline{CK_oU}$DSS TESTS ON EABPL CLAY

FIGURE 5-1



No.	SOIL	W_N %	W_L %	PI %
1	Boston Blue Clay	37	41	21
2	Bangkok Clay	52	65	41
3	Maine Organic Clay	65	65	33
4	EABPL Clay	75±	85±	65±

τ_h = Horizontal Shear Stress

γ = Shear Strain

E_u/S_u VERSUS APPLIED SHEAR STRESS LEVEL FROM $\overline{CK_oU}$DSS TESTS ON SEVERAL NORMALLY CONSOLIDATED CLAYS

FIGURE 5-2

6. FINITE ELEMENT PROGRAM

The finite element program used was FEECON*, an improved version of FEAST-3 (D'Appolonia and Lambe, 1970). The bilinear option of the program was used whereby the stress versus strain behavior of the soil is represented by two straight lines, the break occurring at the shear strength of the soil and the modulus at higher stresses being very small. The program allowed full details of the loading sequence to be modeled, but had to be ammended to allow reevaluation of yielded zones as follows.

The first stage of levee construction involves building the toe dikes (See Fig. 2-1). This probably causes some yielding of the underlying soils. Subsequent construction of the adjacent berms may cause a net <u>decrease</u> in the shear stresses in some of these yielded areas. This represents an <u>unloading</u> of the soil and should be modeled using the initial modulus for the soil and not the yielded modulus. With the original program once an element had yielded, FEECON would treat the element as yielded for all future loading increments, and the yielded modulus would be used. The program was therefore ammended to reevaluate yielded zones before a new increment was applied and put elements back into the initial elastic condition if the new loading would result in a reduction in their shear stresses. An iterative procedure was required to do this, since changing the condition of one element may affect the net change in shear stress on adjacent elements. Thus after any changes were made, the yielded zones had once again to be reevaluated. A maximum of five iterations were allowed and generally a stable arrangement of yielded elements was obtained, as indicated by no change in yielded elements from one iteration to the next. In some cases more than five iterations were required to establish precisely which elements should be yielded.

* Described by Simon, Ladd and Christian (1972)

However, a computer run using only two iterations in which very few stable arrangements of yielded elements were obtained gave very nearly the same final results as the five increment run. Thus it was concluded that the first two iterations accomplished the major purposes of the reevaluation, and since the procedure was expensive in terms of computer time, a maximum of five iterations was used.

FEECON has the capacity to model the embankment stiffness as it is constructed and to crudely model simple shallow excavations. Both capabilities were used. The program is also able to model undrained strength anisotropy by considering the variation in s_u with direction of the major principal stress at failure using the elliptical relationship suggested by Davis and Christian (1971).

7. DEVELOPMENT OF THE FINITE ELEMENT MODEL

The floodwayside deformations of the levees are larger than the landside deformations, which is consistent with the lower stability factors of safety of the floodwayside slopes. For reasons of computer space and economy, FEECON is limited to grids of up to 500 elements, which is ample for most problems. However, owing to the extreme width of the Atchafalaya levees and the need for a fine grid to give good definition of the shapes of deformation, about 400 elements were needed to represent in detail the floodwayside half of the levees. Since this is the most critical region of the embankments, it was decided that only the floodwayside slopes and foundation would be included in the finite element analyses. This means that displacement of the levee centerline will not be exactly modeled, but lateral displacements away from the centerline should be represented in good detail.

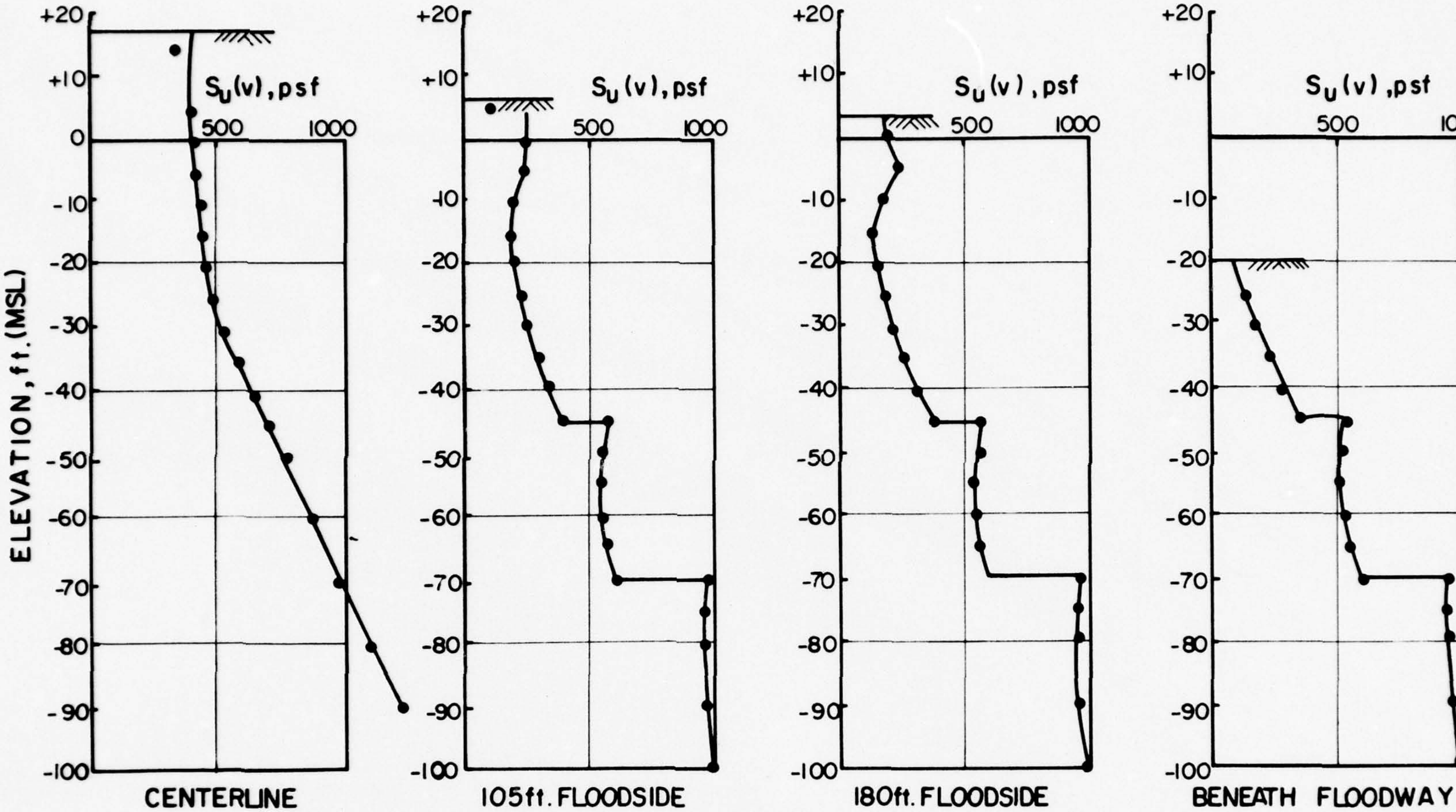
The first step in developing the analysis input was to use the vertical effective stresses and maximum past pressures obtained previously in conjunction with normalized strength parameters to give undrained plane strain active strength profiles shown in Fig. 7-1. These values of $s_u(v)$ were then applied to a floodwayside profile appropriate to both Section II and III to give the contours of undrained strength shown in Fig. 7-2. It should be noted that the horizontal scale shown in Fig. 7-2 is measured from the levee baseline not the centerline. Discontinuities in the strength contours at El. -45 and -70 correspond to the top of the overconsolidated zones.

Figure 7-3 shows the finite element grid. The figure shows the profile of both Sections II and III, but the elements drawn in the levee itself are those used to represent the stiffness of Test Section III.

For each element average values of $\bar{\sigma}_{vo}$, OCR, and undrained strength were then selected, and the elements divided into 23 "soils", each having a constant undrained strength and OCR. Initial stresses for each of these soils were selected on the basis of the values of $\bar{\sigma}_{vo}$ and OCR, using K_o values obtained earlier. The use of K_o implies the neglect of shear stresses due to the slope of the foundation surface. This is another assumption of the analysis.

Selection of a modulus consistent with the average stress level and stress system for each "soil" was then required. This was done by estimating E_u / s_u values, submitting a computer run using these values, and evaluating the final stress level and stress system for each soil. Final values of E_u / s_u were then selected based on the variations with stress level, stress system, and OCR discussed above. The absolute level of the E_u / s_u values was fixed by using 100 for most of the highly stressed soils and referencing all other values to this base value. The foundation soils and the values of E_u / s_u used in the final analysis are shown in Fig. 7-4. Figure 7-5 presents the values of shear modulus G, which resulted from the values shown in Fig. 7-4.

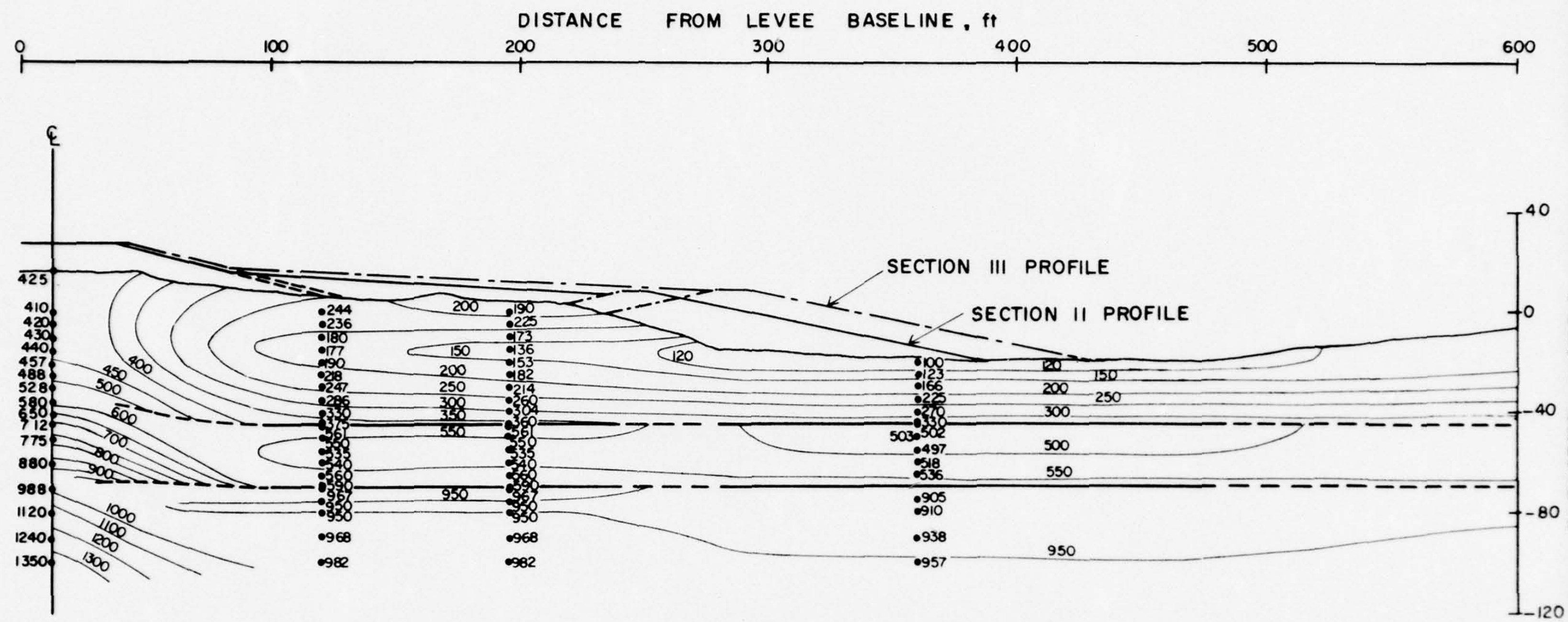
For the levee material itself, a strength of 300 psf was used for hauled fill and 200 psf for the hydraulic and cast-in-place fills, along with E_u / s_u values of 150 and 50 respectively.



Note : S_u (v) =PLANE STRAIN ACTIVE S_u WHERE $S_u = 0.5\,(\sigma_1-\sigma_3)\,f$

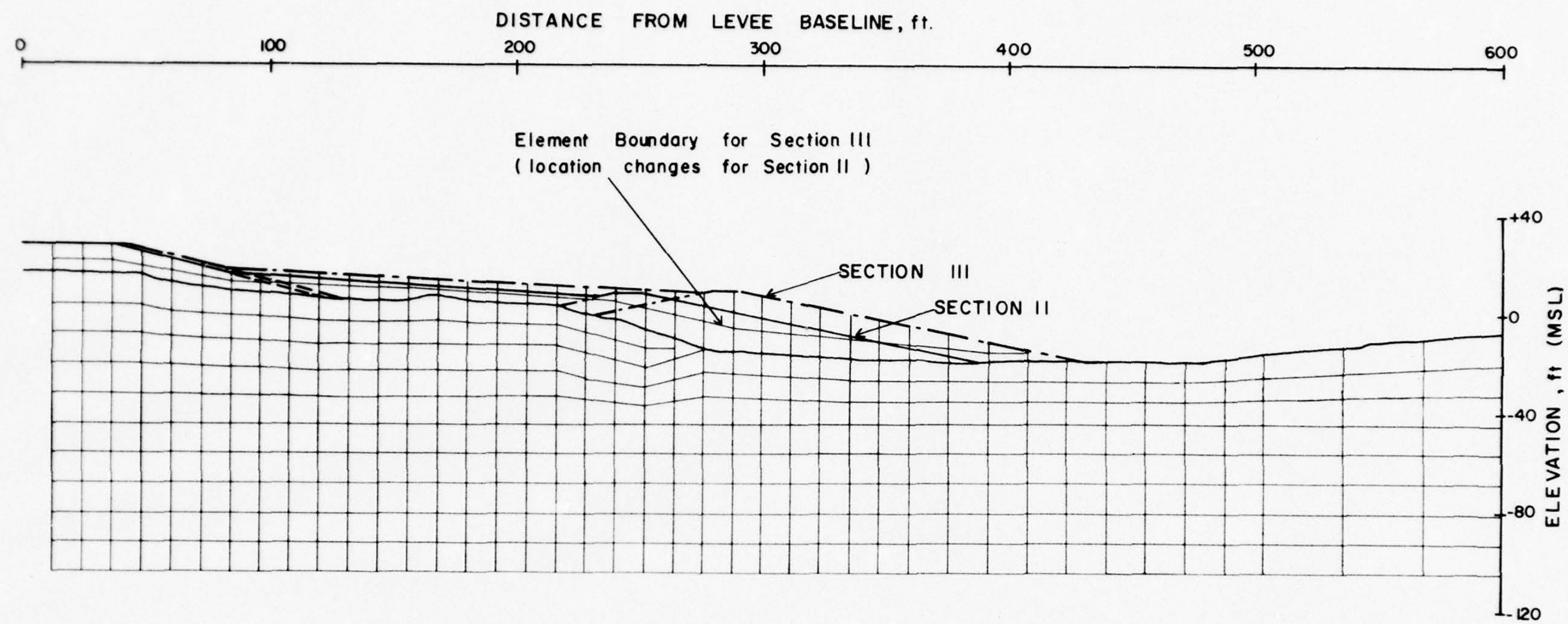
PROFILES OF VERTICAL UNDRAINED STRENGTH, SECTIONS II AND III

FIGURE 7–1



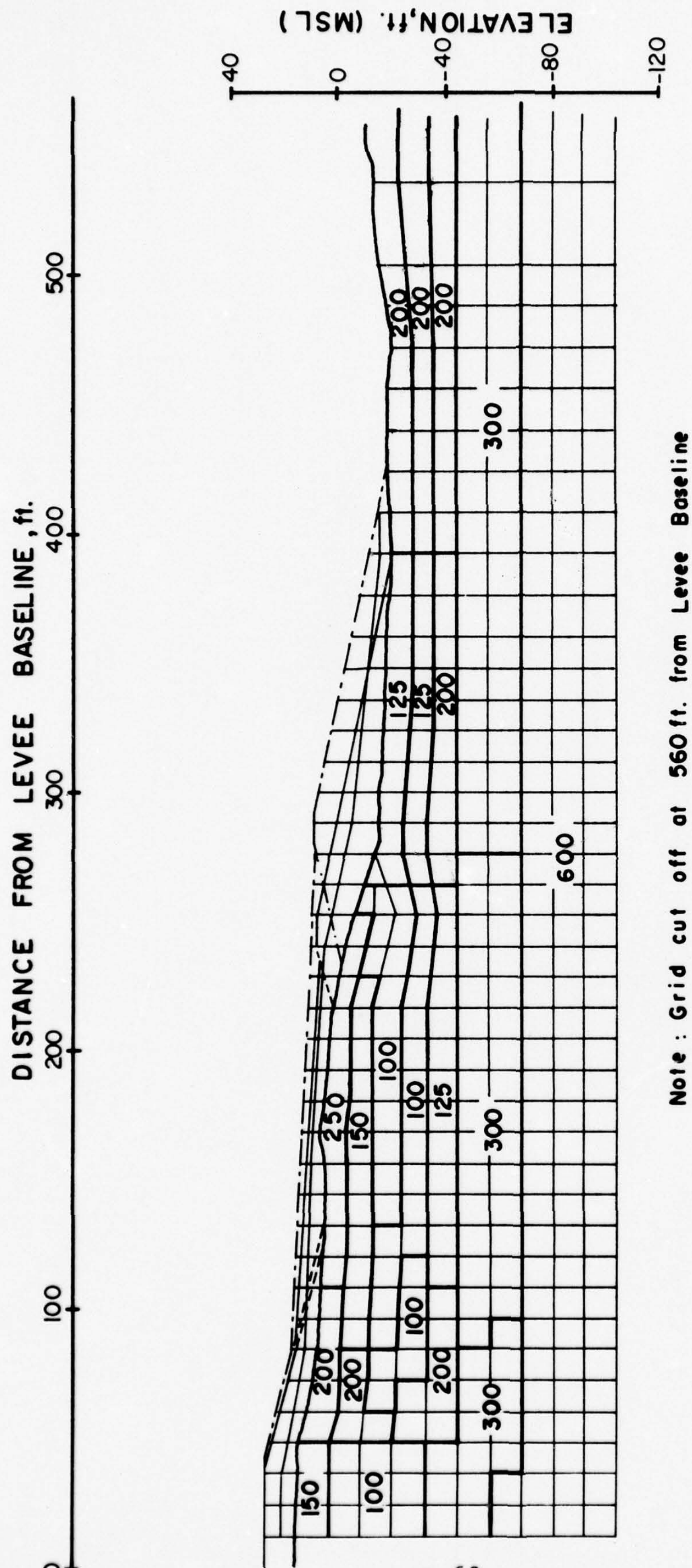
CONTOURS OF VERTICAL UNDRAINED STRENGTH IN THE FOUNDATION FOR SECTIONS II & III

FIGURE 7 —2



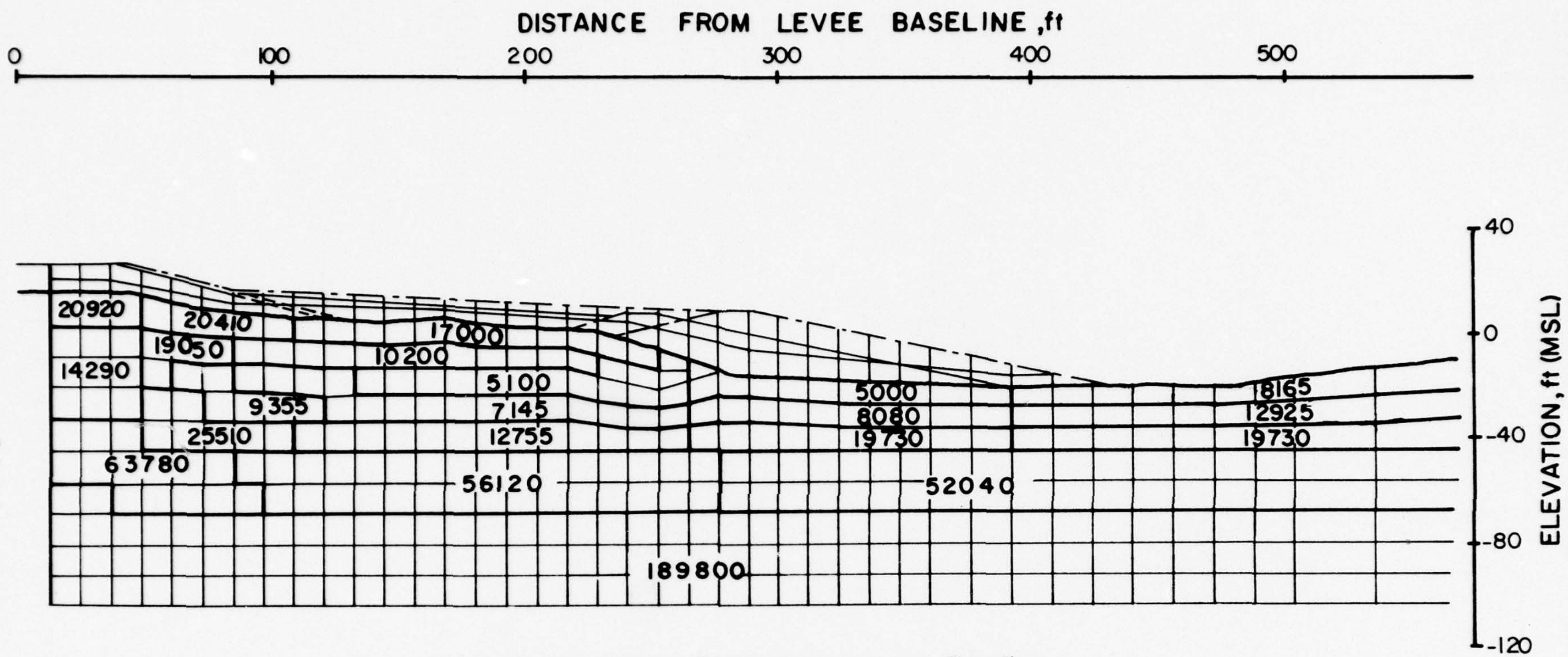
FINITE ELEMENT GRID FOR FEECON

FIGURE 7-3



E_u / S_u VALUES FOR SECTIONS II AND III

FIGURE 7-4



Note : (1) Grid cut off at 560 ft from Levee Baseline

(3) Values of G in PSF

VALUES OF G FOR FOUNDATION SOILS

FIGURE 7-5

8. FINITE ELEMENT RESULTS FOR TEST SECTIONS II AND III

The foundation representation developed in the previous chapter was used for both Test Sections II and III. Section III was analysed first.

8.1 RESULTS FOR TEST SECTION III

The total loads applied during levee construction and the sequence of the loading was evaluated in detail. The loading was applied to the foundation in eleven increments with reevaluation of yielded zones whenever appropriate. Excavation of the opposite floodway bank to obtain borrow material was also represented, based on the as-built drawings. However, this was analysed as two separate increments applied after full levee construction was completed, in order to isolate the resulting deformations and to determine the precise effect of the excavation. In actual fact the excavation occurred during construction of the toe dike.

The results of the analyses are presented in Figs. 8-1 to 8-8. The yielded and highly stressed elements and the lateral deformations at the end of various stages of construction are shown. The amount of levee constructed at each stage is shown by the crosshatched zones. Figures 8-9 and 8-10 present measured lateral deformations at various times after the end of construction, at locations roughly corresponding to the first three locations of deformations shown in Figs. 8-5 to 8-8. The shaded deformations in Fig. 8-9 and 8-10 correspond approximately to the stages of construction represented by Fig. 8-6 and 8-8 respectively. The effect of the excavation was very small.

Table 8-1 lists the predicted lateral deformations at the end of construction at the three offsets closest to the three slope indicators.

8.2 RESULTS FOR TEST SECTION II

Section II was analysed in the same way as Section III, except that the excavation was ignored because the effect had been very small with Section III and the as-built drawings showed the excavation for Section II to be even farther away. The results of the analyses are presented in Figs. 8-11 through 8-16. Once again the levee is crosshatched on each figure to represent the stage of construction. Figures 8-17 and 8-18 show measured deformations and once again deformations are shaded to correspond approximately to the construction stages of Figs. 8-15 and 8-16.

Table 8-2 lists the predicted end-of-construction lateral movements.

TEST SECTION III: END-OF-CONSTRUCTION PREDICTED LATERAL MOVEMENTS

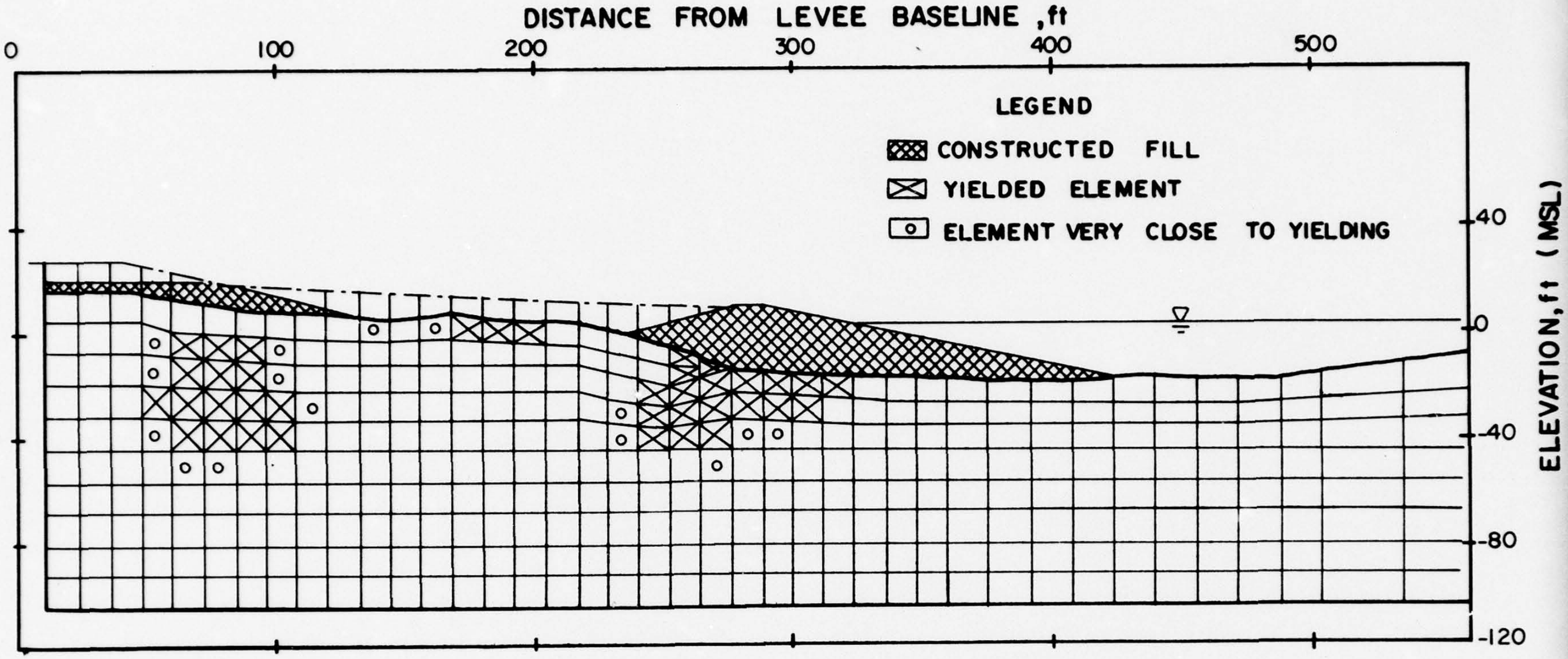
55 Ft Offset from Centerline		120 Ft Offset from Centerline		180 Ft Offset from Centerline	
El. (ft)	Movement (ft)	El. (ft)	Movement (ft)	El. (ft)	Movement (ft)
-105	0	-105	0	-105	0
- 93	.003	- 93	.006	- 93	.006
- 81	.005	- 81	.012	- 81	.011
- 69	.007	- 69	.017	- 69	.016
- 57	.014	- 57	.036	- 57	.031
- 45	.022	- 45	.052	- 45	.046
- 34	.041	- 35	.108	- 35	.096
- 23	.081	- 25	.179	- 26	.193
- 11	.165	- 15	.293	- 16	.333
0	.230	- 5	.333	- 6	.404
+ 11	.312	+ 5	.362	+ 4	.439

Table 8-1

TEST SECTION II: END-OF-CONSTRUCTION PREDICTED LATERAL MOVEMENTS

55 Ft Offset from Centerline		120 Ft Offset from Centerline		180 Ft Offset from Centerline	
El. (ft)	Movement (ft)	El. (ft)	Movement (ft)	El. (ft)	Movement (ft)
-105	0	-105	0	-105	0
- 93	.006	- 93	.010	- 93	.009
- 81	.011	- 81	.018	- 81	.016
- 69	.015	- 69	.026	- 69	.023
- 57	.029	- 57	.050	- 57	.040
- 45	.041	- 45	.071	- 45	.055
- 34	.067	- 35	.139	- 35	.097
- 23	.164	- 25	.232	- 26	.172
- 11	.225	- 15	.330	- 16	.279
0	.271	- 5	.356	- 6	.338
+ 11	.323	+ 5	.374	+ 4	.367

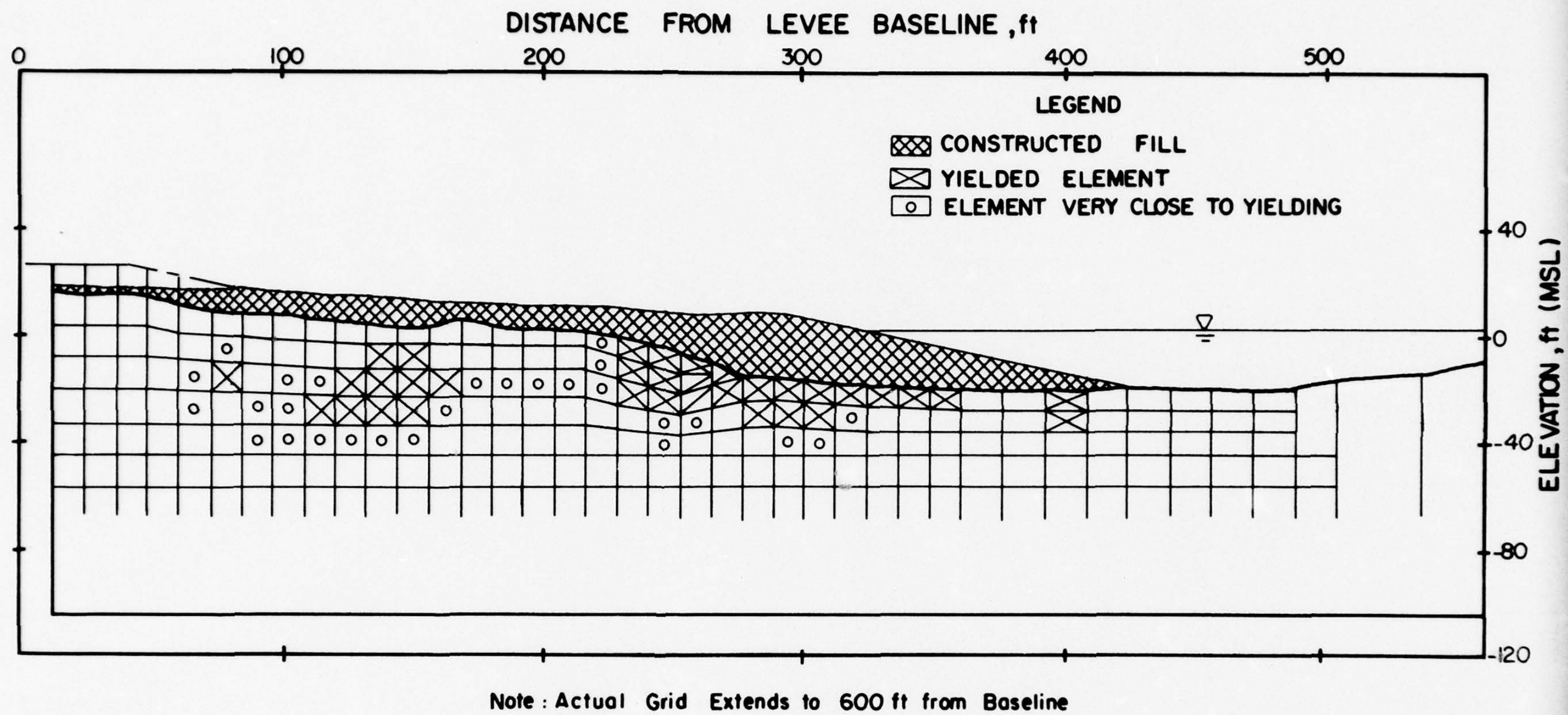
Table 8-2



Note : Actual Grid Extends to 600 ft. from Baseline

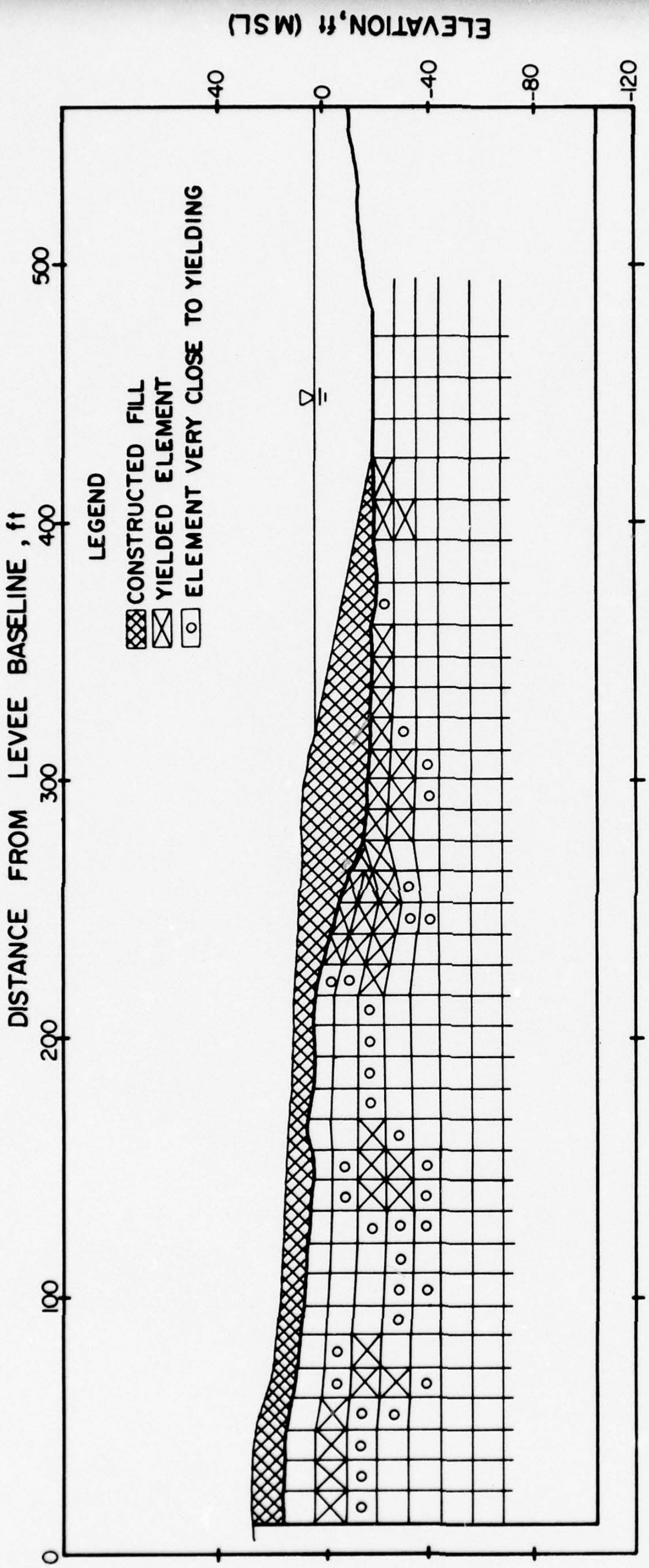
SECTION III: YIELDED ZONES , LOAD STAGE I (RUN 3)

FIGURE 8-1



SECTION III : YIELDED ZONES, LOAD STAGE 2 (RUN 3)

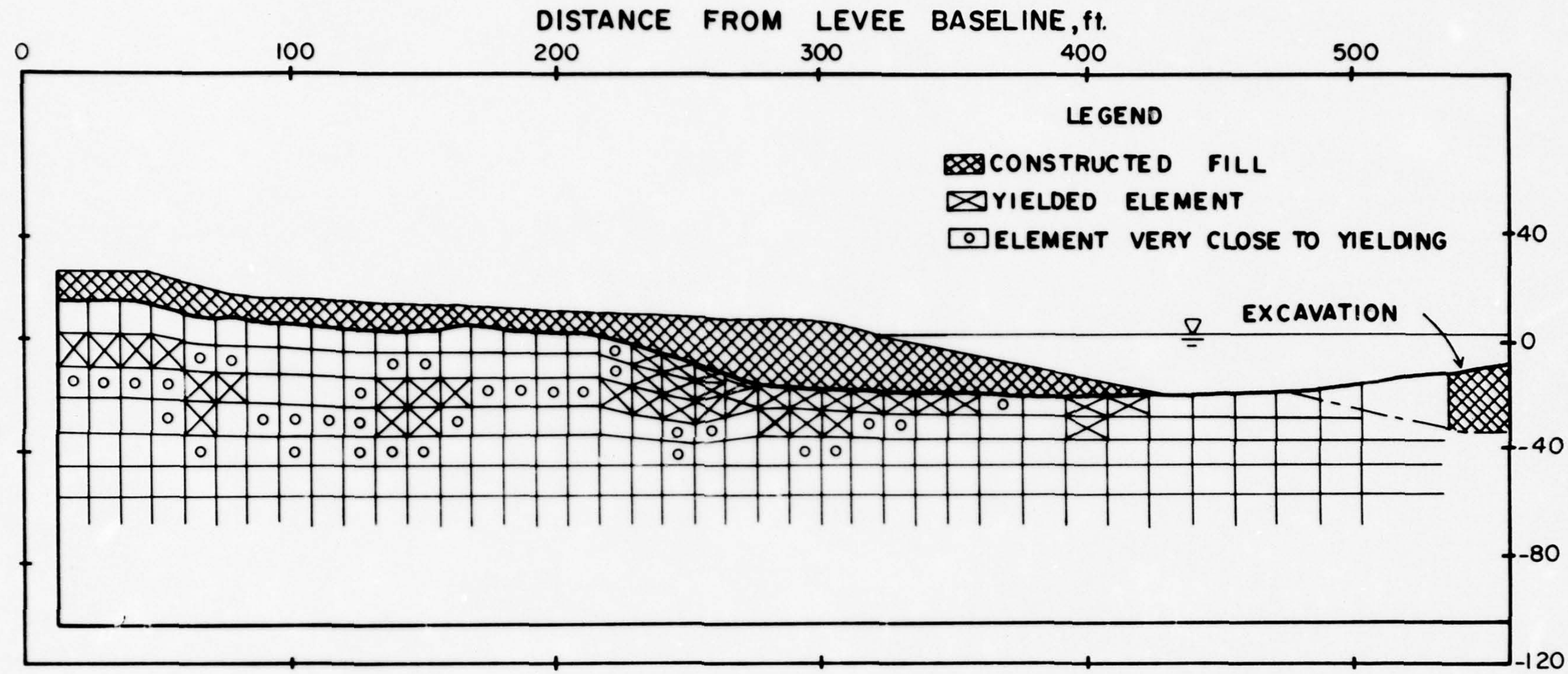
FIGURE 8-2



Note: Actual Grid Extends to 600 ft from Baseline

SECTION III: YIELDED ZONES, LOAD STAGE 3 (RUN 3)

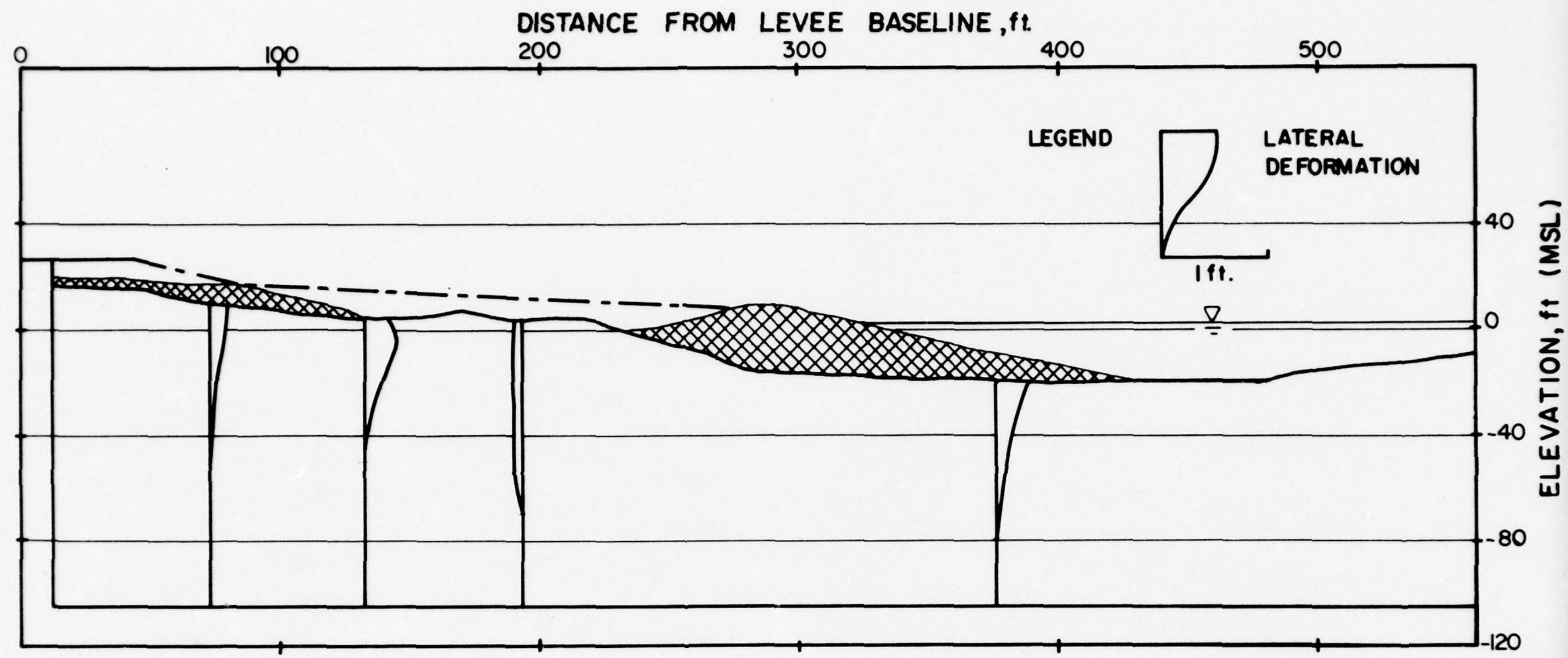
FIGURE 8-3



Note: Actual Grid Extends to 600 ft. from Baseline

SECTION III : YIELDED ZONES, LOAD STAGE 4 (RUN 3)

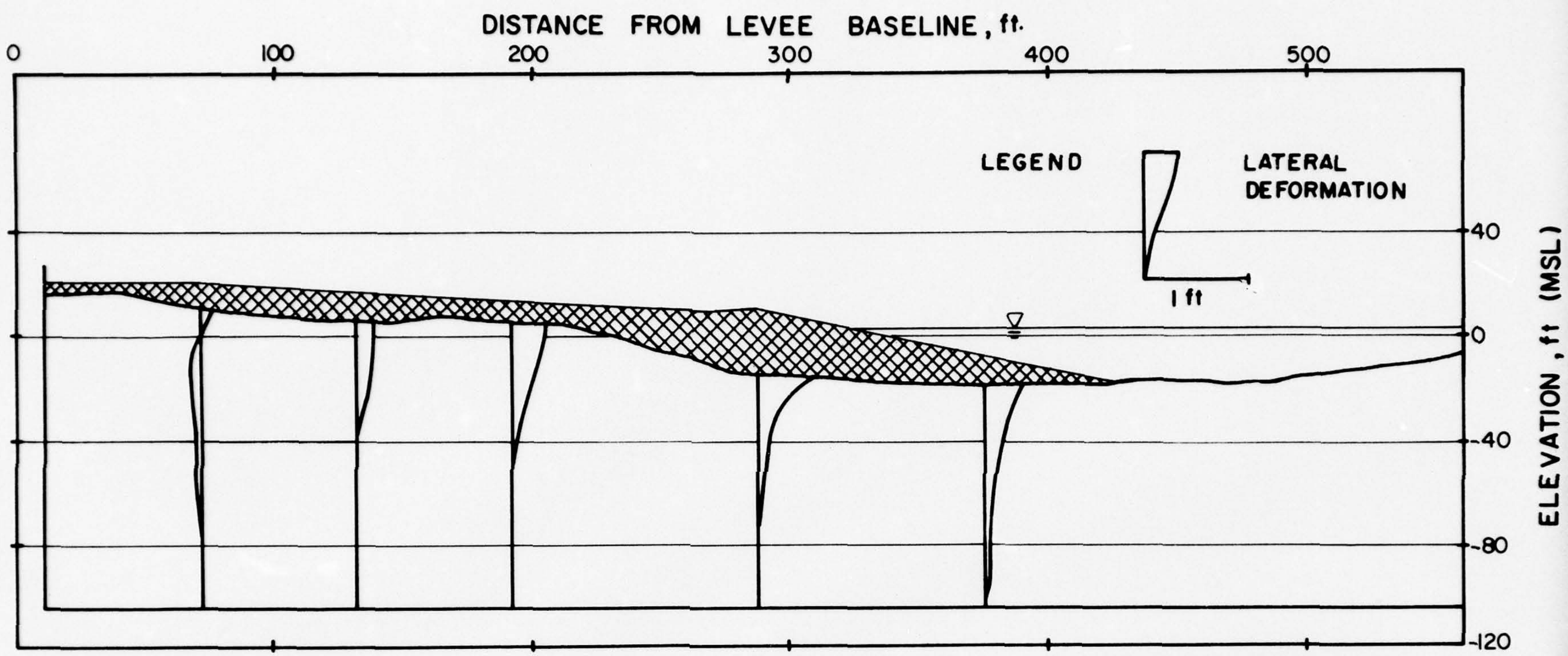
FIGURE 8-4



Note: Actual Grid Extends to 600 ft. from Baseline

SECTION III: LATERAL DEFORMATIONS, LOAD STAGE I (RUN 3)

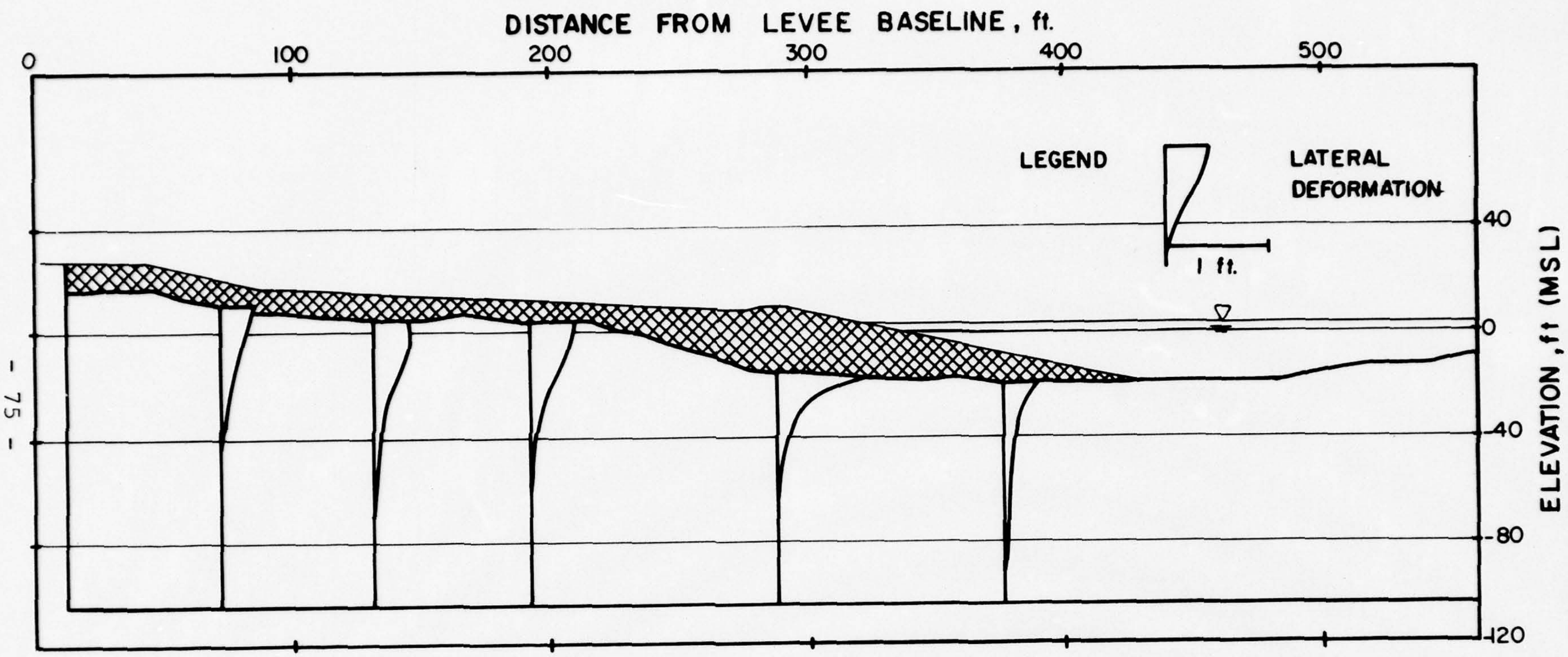
FIGURE 8-5



Note : Actual grid extends to 600 ft from Baseline

SECTION III: LATERAL DEFORMATIONS, LOAD STAGE 2 (RUN 3)

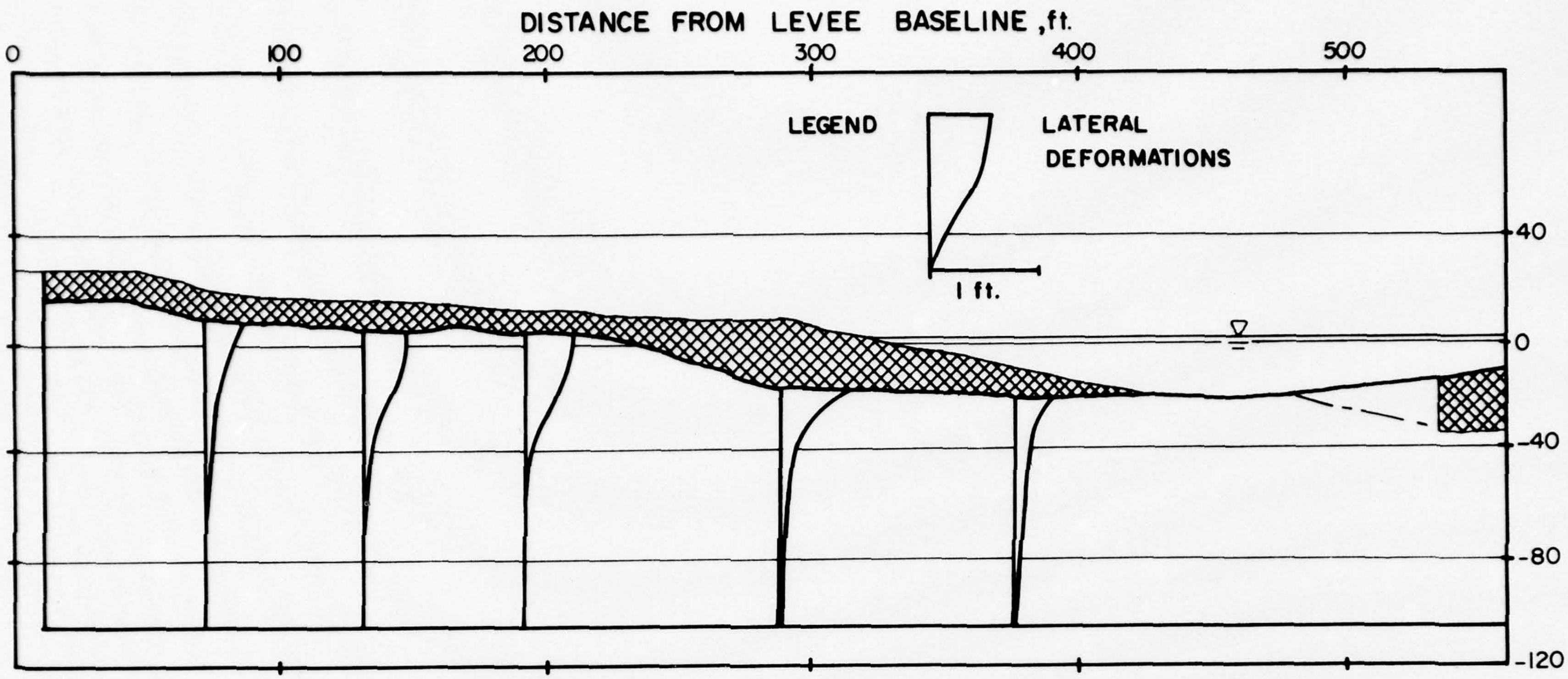
FIGURE 8-6



Note: Actual Grid Extends to 600 ft. from Baseline

SECTION III: LATERAL DEFORMATIONS, LOAD STAGE 3 (RUN 3)

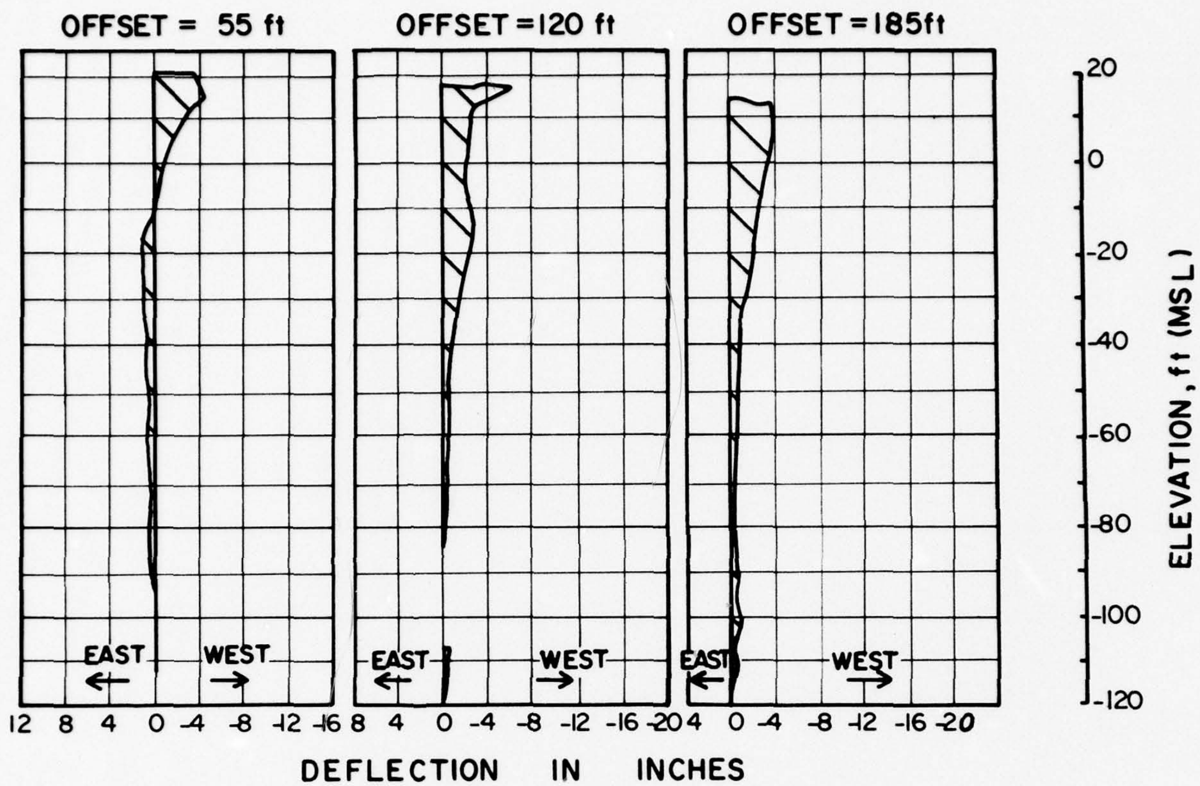
FIGURE 8-7



Note: Actual Grid Extends to 600 ft. from Baseline

SECTION III : LATERAL DEFORMATIONS, LOAD STAGE 4 (RUN 3)

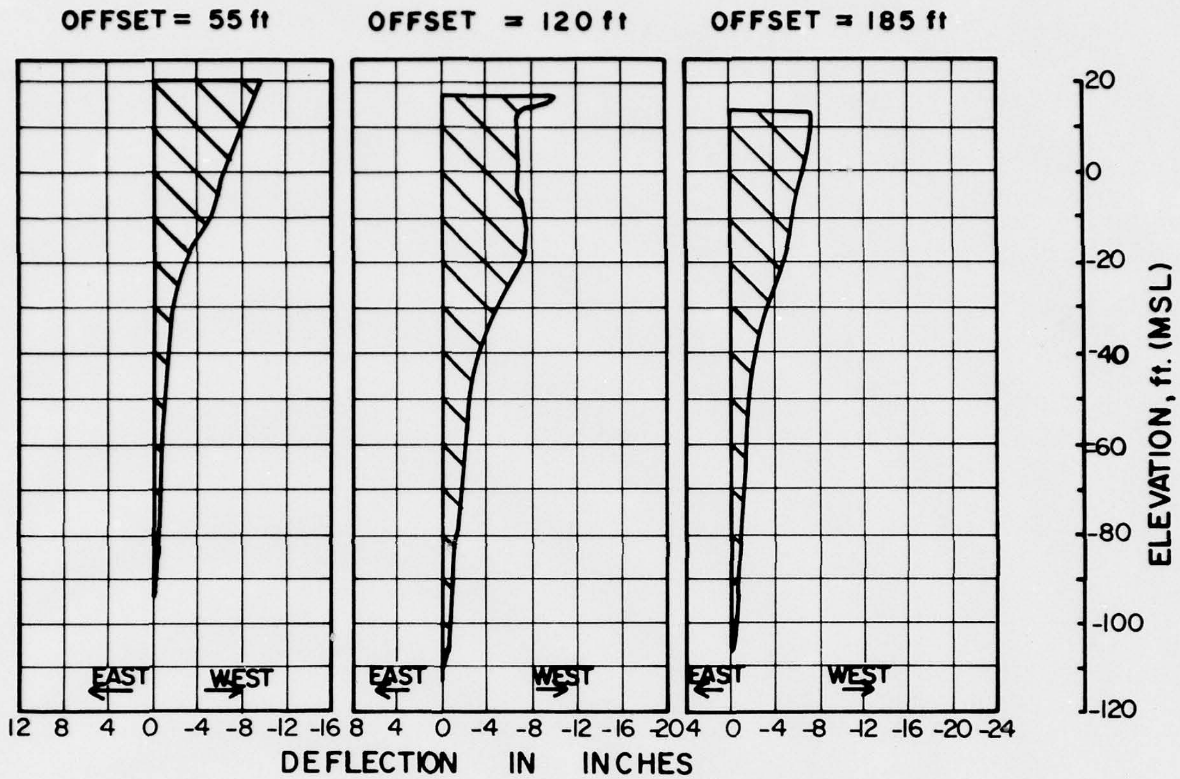
FIGURE 8-8



DEFORMATIONS CORRESPOND APPROXIMATELY TO LOAD STAGE 2 OF FIG. 8-6

SECTION III: MEASURED LATERAL DEFORMATIONS AT LOAD STAGE 2

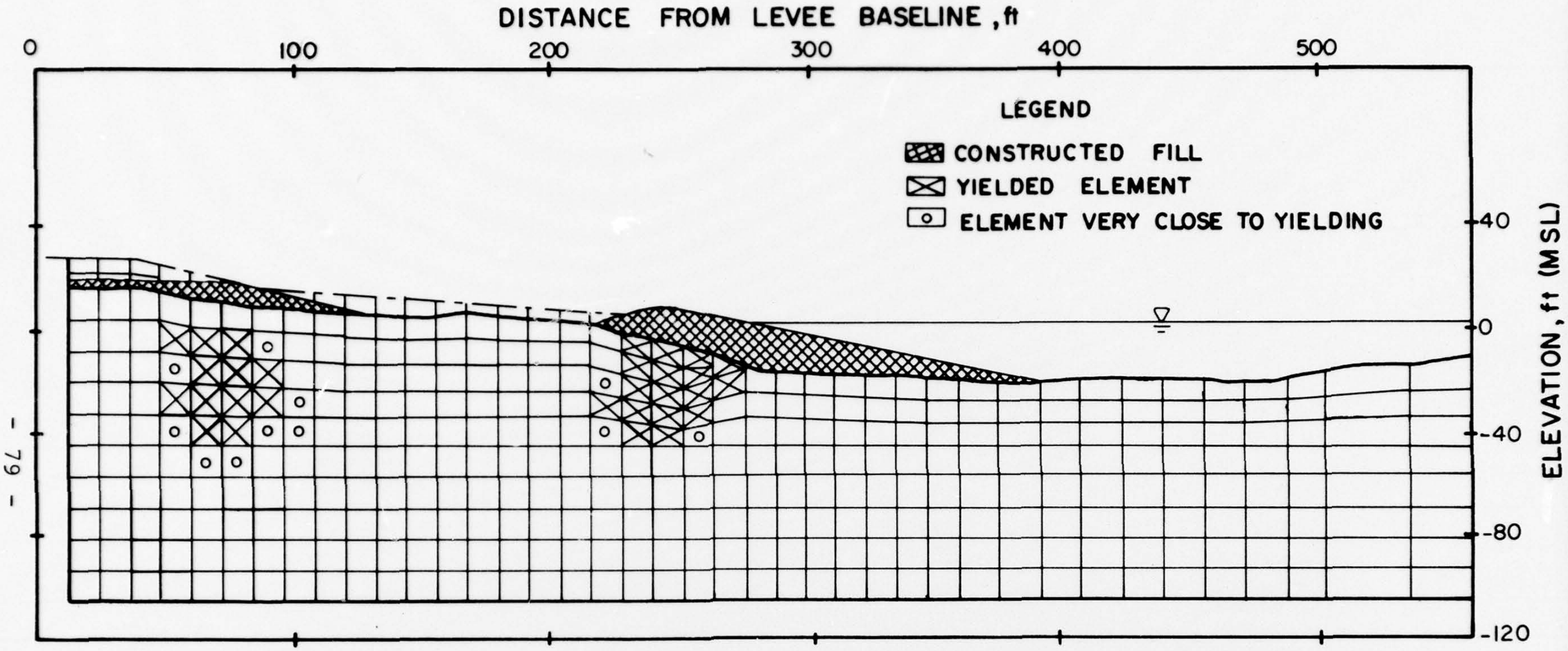
FIGURE 8-9



DEFORMATIONS CORRESPOND APPROXIMATELY TO LOAD STAGE 3 AND 4 OF FIG. 8-8

SECTION III: MEASURED LATERAL DEFORMATIONS AT LOAD STAGE 3 AND 4

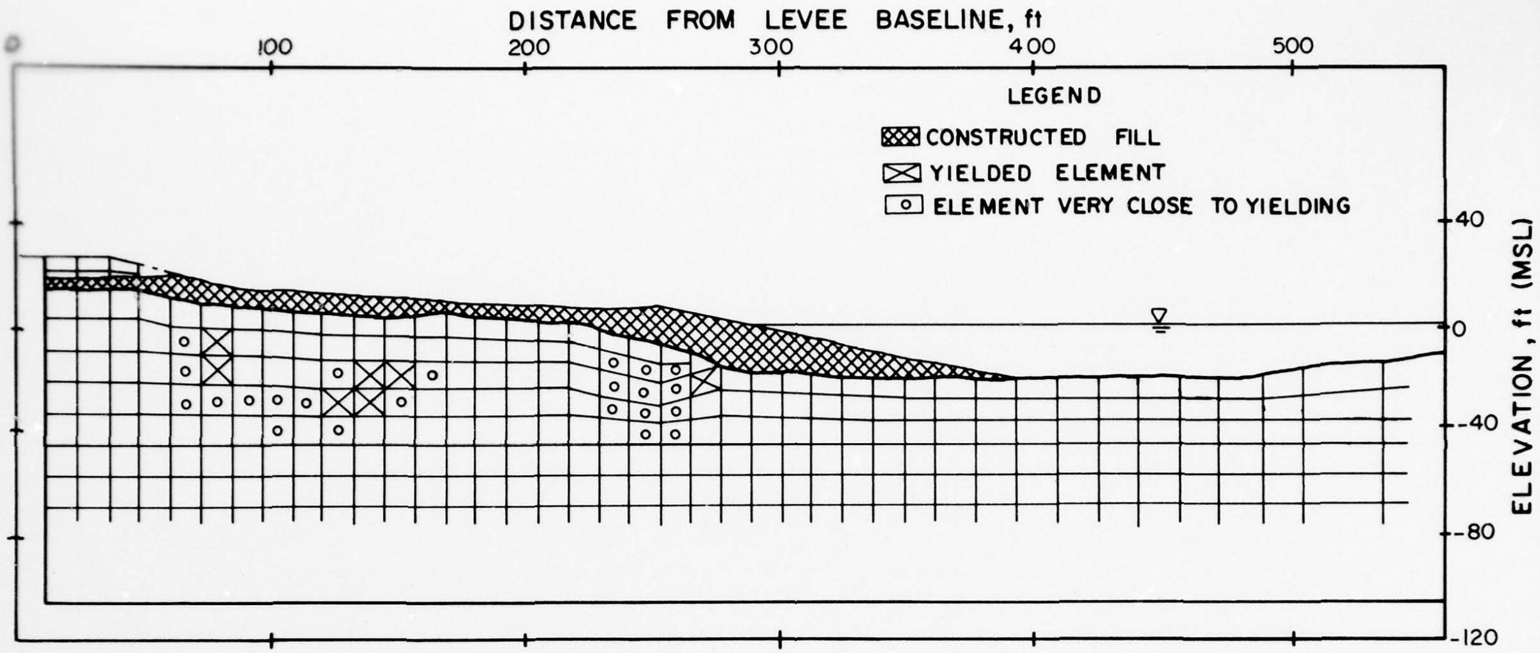
FIGURE 8-10



Note: Actual Grid Extends to 600ft. from Baseline

SECTION II : YIELDED ZONES , LOAD STAGE I (RUN 6)

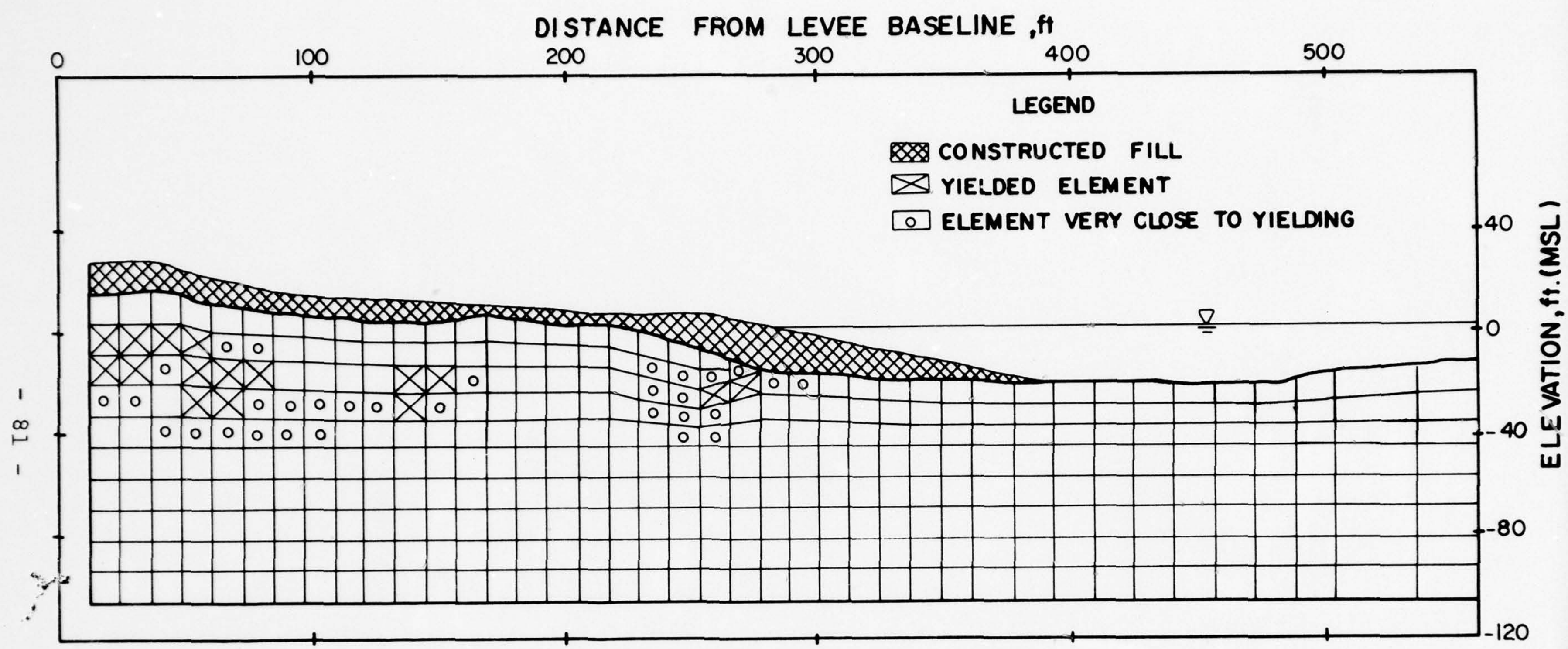
FIGURE 8-11



Note: Actual Grid Extends to 600 ft. from Baseline

SECTION II: YIELDED ZONES, LOAD STAGE 2 (RUN 6)

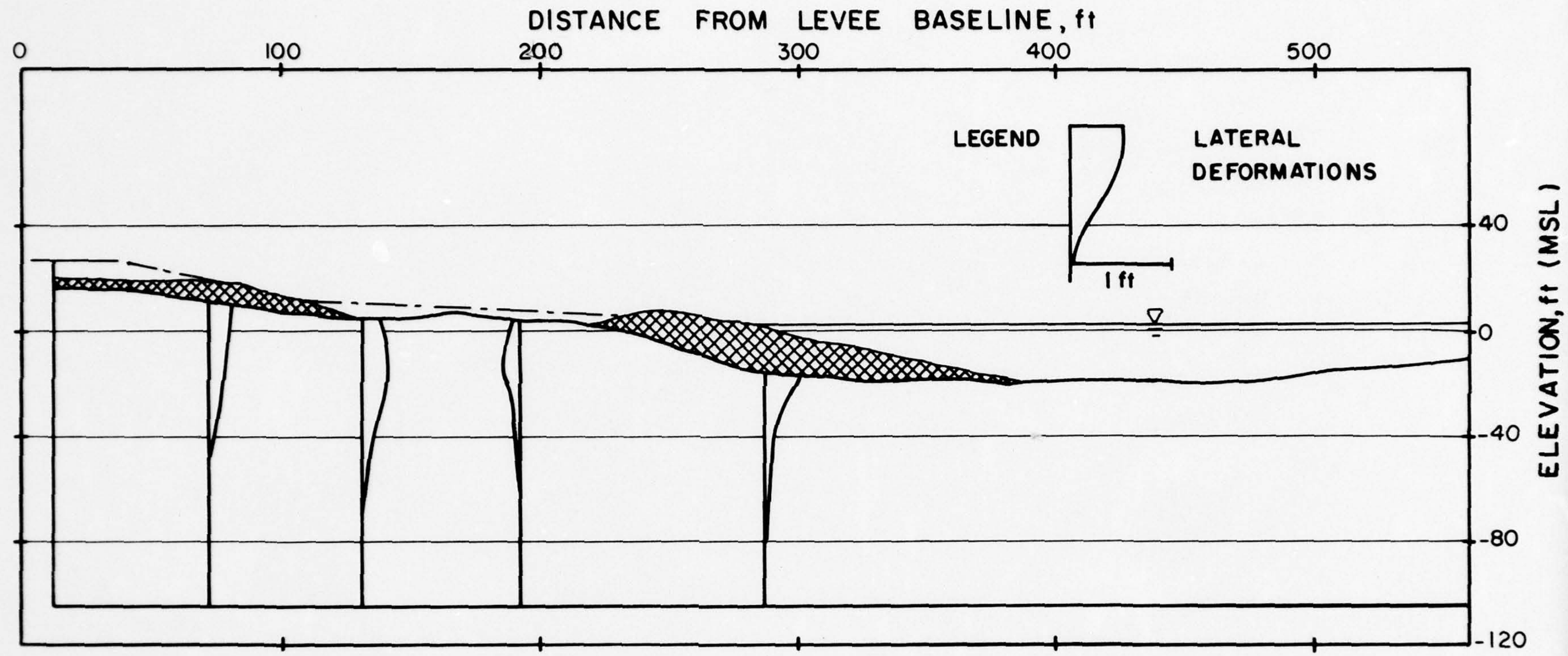
FIGURE 8-12



Note : Actual Grid Extends to 600 ft. from Baseline

SECTION II: YIELDED ZONES, LOAD STAGE 3 (RUN 6)

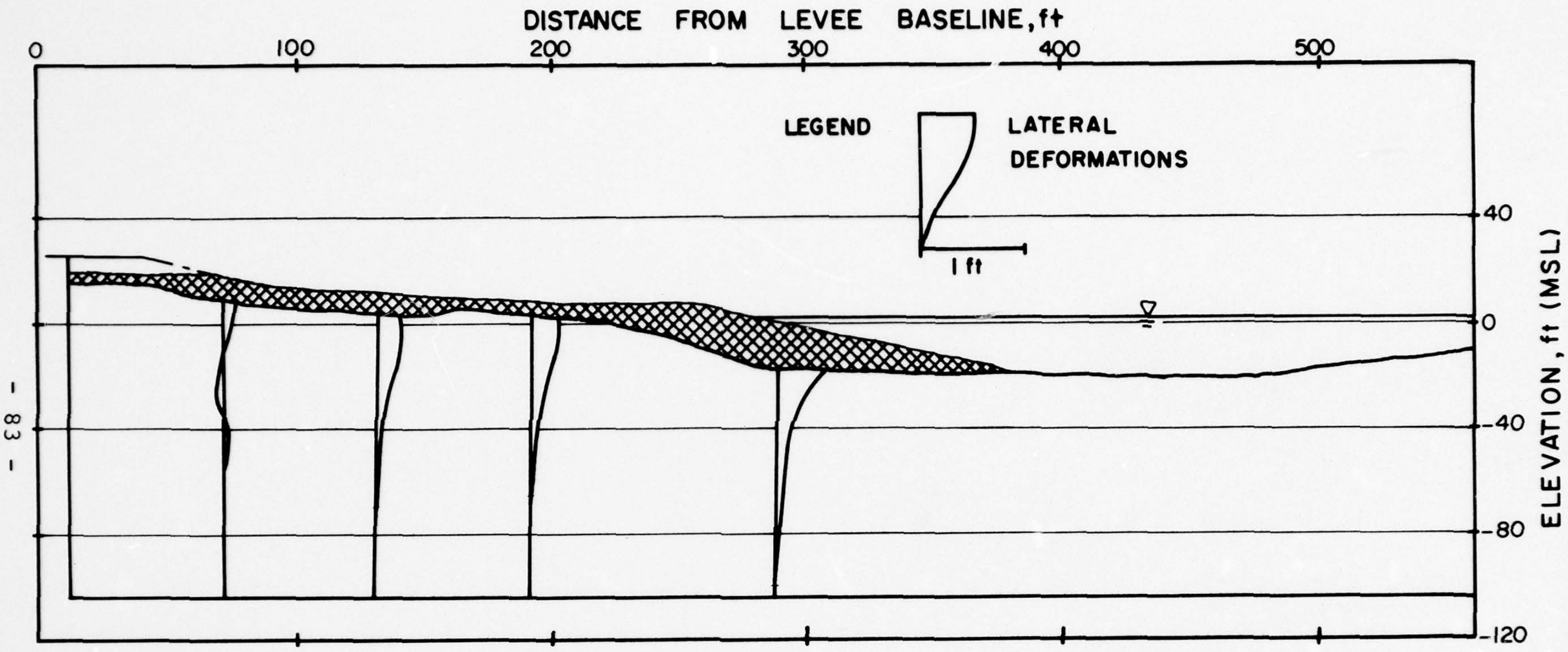
FIGURE 8-13



Note: Actual Grid Extends to 600 ft. from Baseline

SECTION II: LATERAL DEFORMATIONS, LOAD STAGE I (RUN 6)

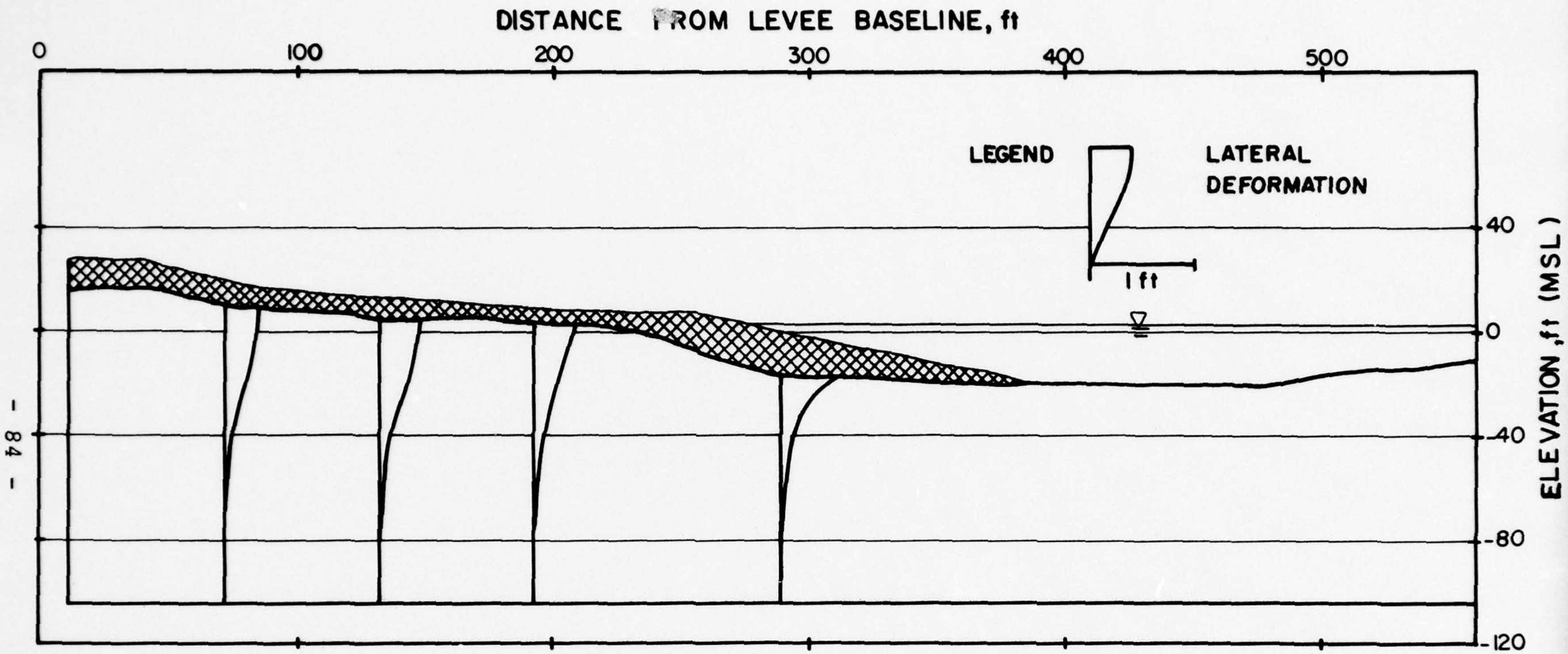
FIGURE 8-14



Note : Actual Grid Extends to 600 ft. from Baseline

SECTION II: LATERAL DEFORMATIONS, LOAD STAGE 2 (RUN 6)

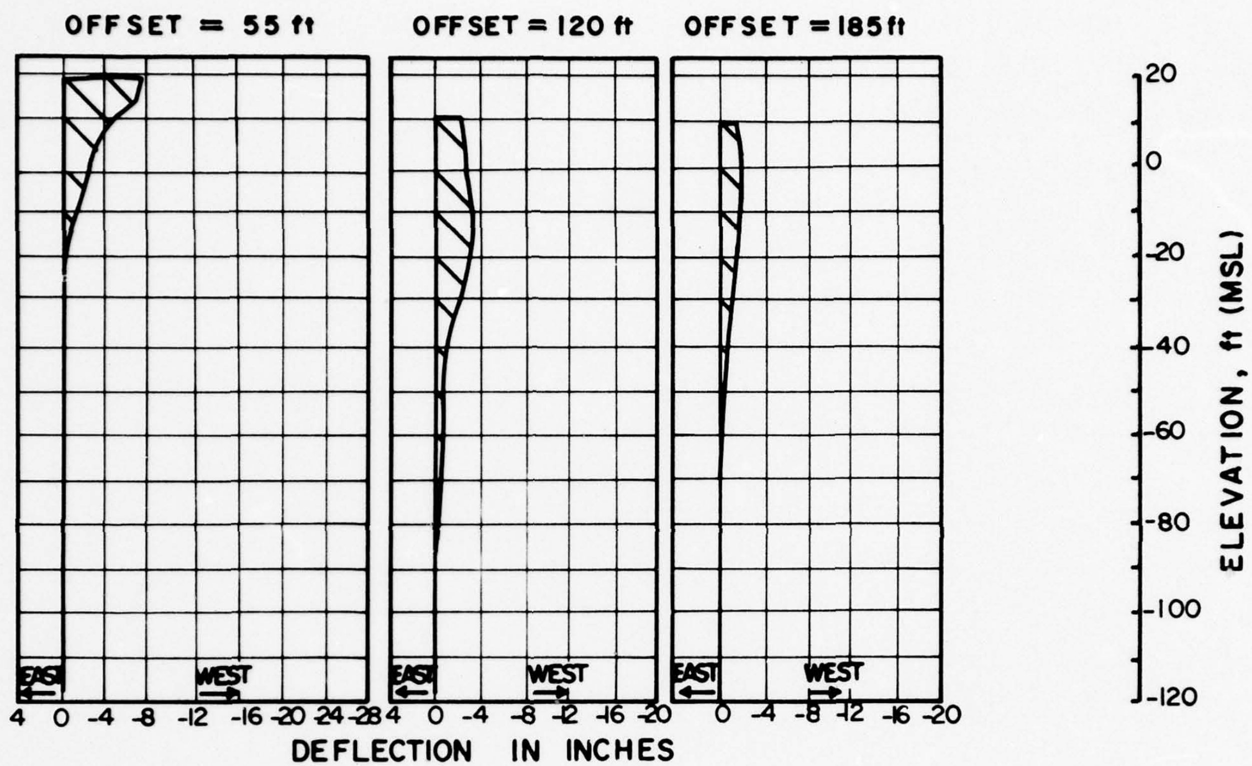
FIGURE 8-15



Note : Actual Grid Extends to 600 ft from Baseline

SECTION II : LATERAL DEFORMATIONS , LOAD STAGE 3 (RUN 6)

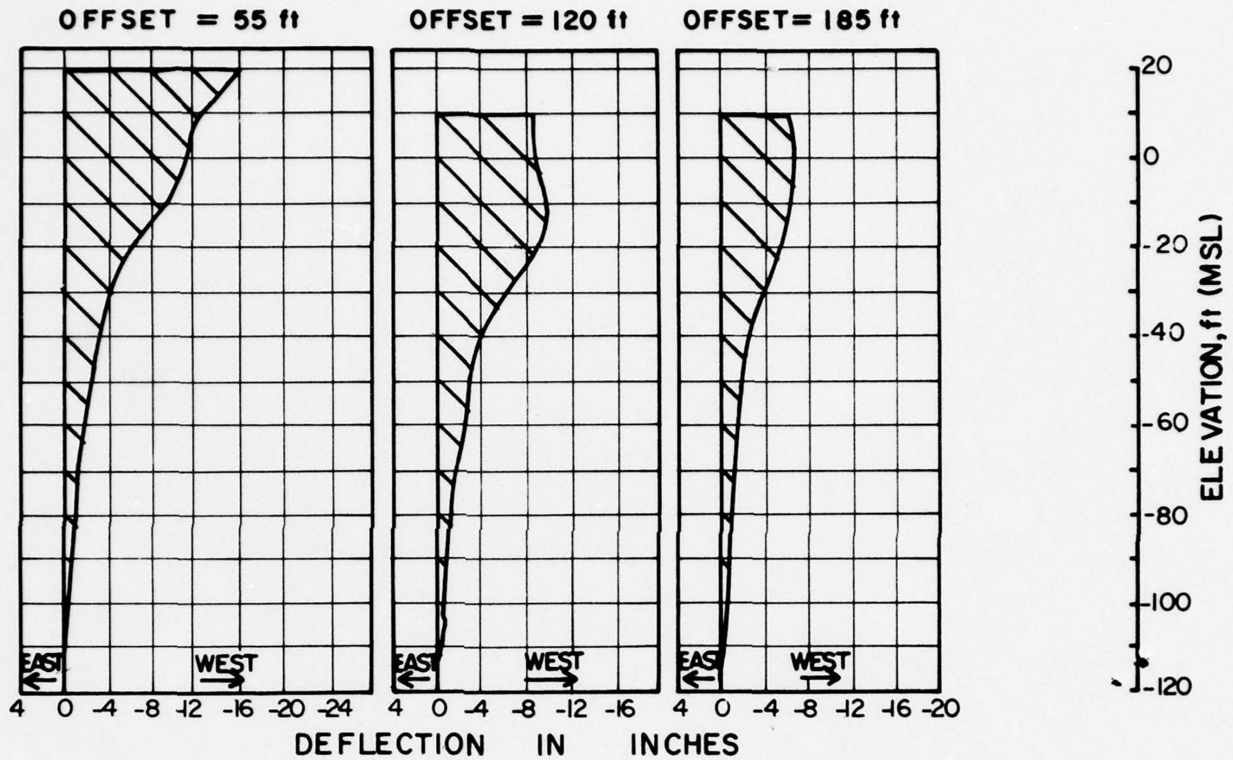
FIGURE 8-16



DEFORMATIONS CORRESPOND APPROXIMATELY TO LOAD STAGE 2 OF FIG. 8-15

SECTION II : MEASURED LATERAL DEFORMATIONS AT LOAD STAGE 2

FIGURE 8-17



DEFORMATIONS CORRESPOND APPROXIMATELY TO LOAD STAGE 3 OF FIG. 8-16

SECTION II: MEASURED LATERAL DEFORMATIONS AT LOAD STAGE 3

FIGURE 8-18

9. EVALUATION OF RESULTS

The chapter dealing with the measured performance of the Test Sections identified the large lateral deformations and particularly the highly stressed zone between El. -20 and -40 as being the most significant characteristics of the levee behavior. This characteristic behavior was the most important aspect of the finite element analyses if the analyses were to give true insight into the performance of the embankment foundation soils. Reference to Figs. 8-8, 8-10, 8-16, and 8-18 shows that in general the shapes of the end-of-construction lateral deformations predicted by the finite element analyses are in good to excellent agreement with those measured, particularly in the case of Test Section III. There is some tendency for the highly stressed zone to extend a little higher than El. -20, but the presence of such a zone is clearly indicated at approximately the correct location. Comparison of the shapes of the predicted and measured deformations before construction of the levee crest (Figs. 8-6, 8-9, 8-15, and 8-17) also shows good agreement, particularly when the fact that some of the deformations were measured before the berms were fully completed is taken into account.

The magnitudes of deformations are of the correct order of magnitude, although they frequently are too small by a factor of two or more. As discussed in Chapter 5, the selection of the absolute level of modulus is largely an empirical process. Since varying the absolute level of modulus will not affect the shapes of deformations and since it is shapes which are most important in this analysis, the present relative values are considered adequate. However, for further analyses, reducing all moduli by a factor of two would more accurately predict the magnitude of the measured deformations. Or more simply, the predicted deformations will be twice as large if the values of shear modulus presented in Fig. 7-5 are cut in half. Figures 9-1 and 9-2 compare measured deformations with predicted deformations increased by a factor of two.

Consideration of the shear stresses at the end of construction (Figs. 8-4 and 8-13) shows good agreement between the highly stressed or yielded areas and both the critical surface locations from stability analyses (performed outside the scope of this contract*) and the zones of large creep deformations.

The results of the analyses appear to be excellent, especially considering the fact that none of the soil parameters were subsequently adjusted in order to achieve even better correlations between predicted and measured performance. However, many assumptions were involved in the analyses and these should now be considered. The major assumptions made were as follows:

(1) The undrained strength measured at normal laboratory strain rates was applied to a field situation with much lower strain rates

(2) Normalized parameters obtained for soils below El. -25 were applied to the very high water content layer above El. -25

(3) Initial horizontal shear stresses due to the slope of the ground surface at the floodway and the presence of the original levee were neglected

(4) Only the floodwayside halves of the levees were analysed, with the assumption of symmetry at the levee centerline.

In addition there were many smaller assumptions made in selection of moduli and in dividing the foundation into "soils" of uniform properties, plus the standard assumptions implied by the use of bilinear stress versus strain relationships.

Considering the major assumptions in turn:

(1) The yielded and highly stressed zones predicted by the analyses appear to be consistent with the measured behavior and the location of critical stability surfaces. If lower

* Presented in EABPL Memorandum No. 6 "Stability Analyses: Test Sections II and III", by R. Foott and C. Ladd, June 1971.

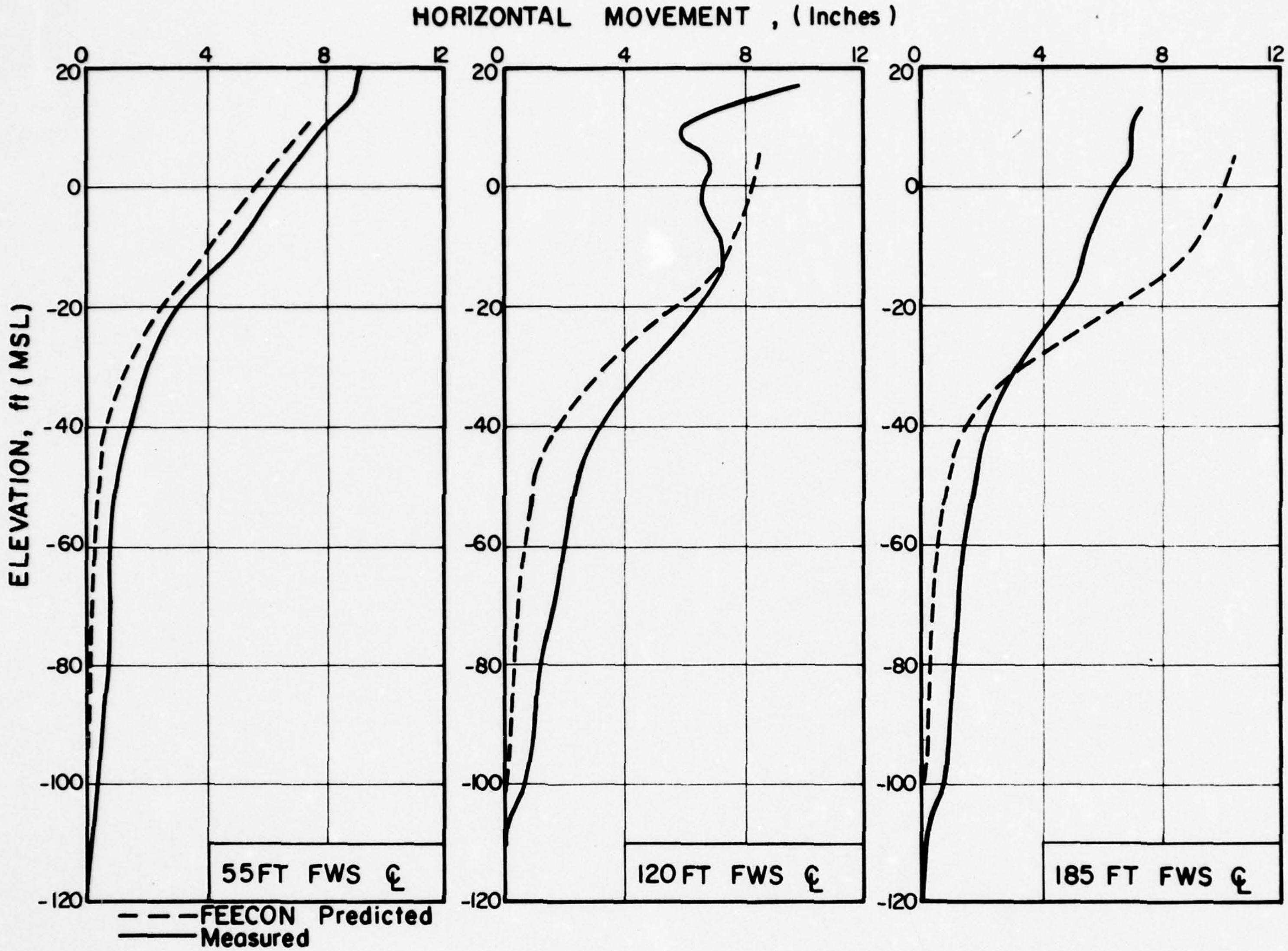
strengths were used, the yielded areas would increase very rapidly, as would deformations, and the results of the analyses certainly would not be as good. Thus it appears as though the assumption was appropriate, although there is no really sound way of proving that the selected values of s_u are in fact the actual in situ strengths.

(2) It is unlikely that the parameters for the high water content layer above El. -25 could have been much different from the values used, so this assumption appears reasonable.

(3) Initial shear stresses would exist and from slope stability analyses it is likely that they would have been largest in the zones which were highly sheared at the end of construction. Thus they would tend to increase the predicted yielding and deformations. However, their effect is difficult to quantify and would be interrelated with assumption (1) above. Thus collectively these two assumptions have been accepted on the grounds that the analyses seem to give correct results, but without any really sound further justification.

(4) Clearly the levees are not symmetrical, and so the assumption is in error. However, the inaccuracies introduced will decrease with increased distance from the centerline. Furthermore, the improved definition of the deformation patterns obtained by modeling only one half of the levee was considered an overall benefit.

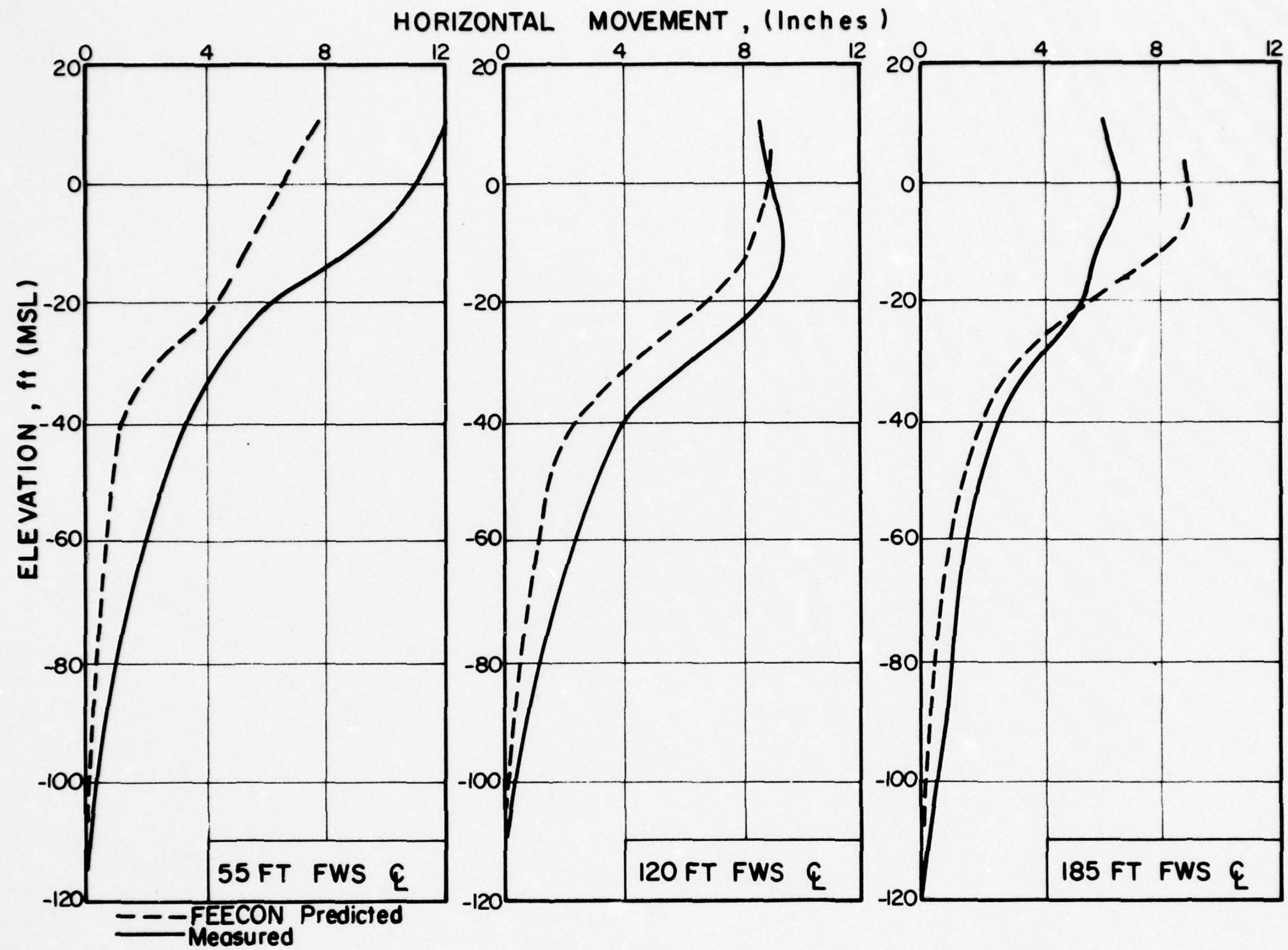
Despite all the assumptions, the authors feel that the finite element analyses have produced a very acceptable representation of the most important characteristics of the end of construction levee foundation behavior. While it would be difficult to predict a priori what the effect of some of the assumptions would be, it seems that: the assumptions were in fact sound; or errors in the assumptions were self compensating; or the basic mechanisms controlling the levee behavior were so important that the characteristic behavior was not obscured by the assumptions.



Note: Predicted deformations with decreased modulus –see p.87

TEST SECTION III – PREDICTED AND MEASURED LATERAL DEFORMATIONS AT END OF CONSTRUCTION

FIGURE 9–1



Note: Predicted deformations with decreased modulus – see p.87

TEST SECTION II – PREDICTED AND MEASURED LATERAL DEFORMATIONS AT END OF CONSTRUCTION

FIGURE 9-2

10. CONCLUSIONS AND RECOMMENDATIONS

It is tentatively suggested that the behavior of the Test Sections can be explained as follows.

When loaded, the foundation soils tend to be displaced away from the centerline. Below El. -45 approximately, there is a layer of overconsolidated material which is stiffer and stronger than the overlying soils. Consequently the deformations below this level are quite small. This layer also causes a concentration of shear stresses in the soil above it up to El. -20. This fact causes the characteristic pattern of the end of construction deformations.

The Atchafalaya soils are very plastic and exhibit large rates of undrained creep. As the stress level becomes high, the creep rate of the soil increases rapidly. Thus the soil above the overconsolidated layer up to El. -20, which is the most highly stressed zone, is also the region which experiences the largest creep deformations.

This tentative explanation of the performance of the levees does agree with the measured field data, and thus the first area of further study should be directed at proving or disproving its validity. Some further work with the finite element analysis, particularly to the landside of the levees, plus a meticulous evaluation of the pore pressure measurements would be a valuable contribution. Undrained creep analyses aimed at explaining the long term lateral deformations of the levee foundation soils, already well advanced, should be completed. Finally, the long term settlements due to consolidation should be thoroughly investigated. Preliminary studies indicate a major discrepancy between measured rates of consolidation (even after accounting for lateral deformations) and those predicted from the results of laboratory tests.

Once armed with a thorough and detailed understanding of the behavior of levee foundation soils, the really important problem of realistically assessing alternative designs can be tackled. The computer modeling techniques developed for the Test Sections could play a valuable part in this process.

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APPENDIX A: NOTATION

CU	Consolidated-undrained shear test
$\overline{CK_oU}$	CU test with K_o consolidation and measurement of pore pressures
$\overline{CK_oU}$DSS	$\overline{CK_oU}$ direct-simple shear test
E_u	Undrained Youngs modulus prior to yielding
FS	Factor of safety
G	Undrained shear modulus
K_o	Coefficient of lateral earth pressure at rest
OCR	Overconsolidation ratio
PI	Plasticity index
R	CU shear test with isotropic consolidation
s_u	Undrained shear strength
$s_u(v)$	Vertical s_u $(\sigma_1 = \sigma_{vf})$
t_c	Time of consolidation
UU	Unconsolidated-Undrained shear test
w_l	Liquid limit
w_n	Natural water content
w_p	Plastic limit
ε_v	Vertical strain
σ_1	Major principal total stress
σ_3	Minor principal total stress
σ_{1f}	σ_1 at failure
$\overline{\sigma}_{vc}$	Vertical consolidation stress
σ_{vf}	Total vertical stress at failure

σ_{vh}	Total horizontal stress
$\bar{\sigma}_{vm}$	Maximum past pressure
$\bar{\sigma}_{vo}$	In situ vertical effective stress
ϕ	Total stress friction angle
τ_h	Horizontal shear stress in a $\overline{CK_oU}$DSS test
γ	Shear strain

APPENDIX B: REFERENCES

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13. ABSTRACT

Levee construction in south-central Louisiana has proven to be very time consuming and expensive because of excessive movements within the soft, highly plastic, thick deposits of the foundation soils. Field measurements from three well instrumented test sections constructed along the East Atchafalaya Basin Protection Levee (EABPL) have led to the belief that undrained shear strains and long term lateral creep deformations are major factors contributing to the very large settlements of the levees. This report presents the results of finite element predictions of the end of construction lateral deformations on the floodwayside of the EABPL Test Sections II and III. The M.I.T. computer program used for the analyses, which realistically modeled the actual sequence of construction, requires input of the following soil parameters for the foundation soils: (1) the initial vertical and horizontal stresses; (2) the undrained shear strength as a function of the orientation of the major principal stress at failure; and (3) a bilinear stress-strain relationship, and particularly the undrained shear modulus prior to yielding. The first step in selection of the soil parameters was a thorough investigation of the stress history of the foundation soils, and especially the in situ maximum past pressure. This study included a detailed consideration of the geology of the area, values of maximum past pressure determined from oedometer tests, the results of field vane tests, and variations in water content and plasticity with depth. A principal conclusion from this work, which proved to be very significant, was the discovery of a highly precompressed zone below El. -45 ft. The stress-strain-undrained strength properties of the foundation clay were investigated via a program of consolidated-undrained triaxial, plane strain, and direct-simple shear tests performed on samples K_o consolidated beyond the in situ stresses. The resulting data were normalized and plotted versus overconsolidation ratio and/or shear stress level. The normalized soil parameters, combined with the stress history developed for the foundation clay and a knowledge of zones of highly stressed soil, were then used to select the soil properties required for the analyses. A significant finding was the fact that the Atchafalaya clays have an unusually low shear modulus at stress levels approaching failure. Only one set of predictions were made for each of the test sections. The results, presented in Chapter 8, showed good to excellent agreement between the predicted pattern of lateral deformations and the measured field performance. Most significantly, the computer analyses predicted that the largest shear strains would occur in the zone between about El. -45 and -20 ft. It was concluded that the computer model and soil parameters that were developed represent a major advance in understanding the fundamental nature of the long term lateral deformations that have proven so detrimental to the performance of the levees.

14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Atchafalaya Levee System						
Clays						
Embankment foundations						
Finite element method						
Levee construction						
Mathematical models						
Soil properties						
Soil stresses						